

FINAL REPORT

March 30, 1994

STRENGTH EVALUATION AND STRUCTURAL ANALYSIS INVESTIGATION

OF

**UNITED STATES FEDERAL BUILDING
517 GOLD AVE. SW, ALBUQUERQUE, NEW MEXICO**

BUILDING NO.: NM0024ZZ
PROJECT NO.: ZTX00210

FOR

**GENERAL SERVICES ADMINISTRATION
REGION 7**

819 Taylor Street
Fort Worth, Texas 76102

REAVELEY ENGINEERS & ASSOCIATES

1515 SOUTH 1100 EAST
SALT LAKE CITY, UTAH 84105
PHONE: (801) 486-3883, FAX: (801) 485-0911

PARTICIPANTS

The architectural / engineering team for this project is listed below:

ARCHITECT:

Naylor Wentworth & Associates, Inc.
175 West 200 South, Suite 1000
Salt Lake City, Utah 84101
(801) 355-5959
(801) 355-5960 Fax

STRUCTURAL:

Reaveley Engineers & Associates, Inc.
1515 South 1100 East
Salt Lake City, Utah 84105
(801) 486-3883
(801) 485-0911 Fax

DISCLAIMER

The following report is an investigative study only. It is not a construction contract document and must not be used as such. Naylor Wentworth & Associates, Inc. and/or Reaveley Engineers and Associates, Inc. accepts no responsibility for rehabilitation work using this document. Any attempt to rehabilitate the United States Federal Building (Building No. NM0024AA, Project No. ZTX00210) at 517 Gold Ave. SW, Albuquerque, New Mexico using this document will be done so at the owners risk.

INDEX

	<u>PAGE</u>
PARTICIPANTS	ii
DISCLAIMER	iii
INDEX	iv
STRENGTH EVALUATION AND STRUCTURAL ANALYSIS INVESTIGATION	
EXECUTIVE SUMMARY	1
PURPOSE AND SCOPE	4
BUILDING DESCRIPTION	5
FIELD INVESTIGATION	7
EVALUATION AND FINDINGS	10
REVIEW OF STRUCTURAL FLOOR ANALYSIS	18
REVIEW OF SEISMIC STUDY	21
SEISMIC ANALYSIS & STRENGTHENING	26
STRENGTHENING MEASURES	28
PRIORITIZATION OF STRENGTHENING MEASURES	33
COST ESTIMATES	34
COST BASIS	36
LIMITATIONS	36

APPENDIX INDEX

A-1	SCOPE OF WORK	9-pages
A-2	STRUCTURAL SUPPORT DATA	
	CONCRETE TEST RESULTS	
	STRENGTH TESTS	7-pages
	PETROGRAPHIC EXAMINATION REPORT BY HI TECH CONSULTING	15-pages
	PETROGRAPHIC EXAMINATION REPORT BY CONSTRUCTION TECHNOLOGY LABORATORIES	17 pages
A-3	COST ESTIMATES	
	REINFORCE FLOORS WITH NEW CONCRETE CAPITALS	3-pages
	REINFORCE FLOORS WITH NEW STEEL COLLARS	3-pages
	ADD NEW TOPPING TO REDUCE FLOOR DEFLECTIONS AND REINFORCE FLOORS WITH NEW CONCRETE CAPITALS	3-pages
	SEISMIC UPGRADE	4-pages

STRENGTH EVALUATION AND STRUCTURAL ANALYSIS INVESTIGATION

OF

UNITED STATES FEDERAL BUILDING

517 GOLD AVE. SW

ALBUQUERQUE, NEW MEXICO

BUILDING NO.: NM0024ZZ

PROJECT NO.: ZTX00210

EXECUTIVE SUMMARY

The purpose of the Strength Evaluation and Structural Analysis Investigation of the Albuquerque, New Mexico Federal Building at 517 Gold Ave. SW was to determine live load capacities for each of the floor slabs, determine the cause of the deflections in the concrete floor slabs, and perform a preliminary seismic study of the building.

The Albuquerque Federal Building is an eight story concrete building with a full basement below grade. The building is rectangular in configuration and is 300 feet long by 102 feet wide. Floor to floor heights are 11'-0". The structural system of the building consists of reinforced concrete flat slabs at the floors and roof. The flat slabs are 8" thick with drop heads at columns. The drop heads are typically 8'-4" x 8'-4" x 4" deep at the interior columns. The flat slabs are supported by reinforced concrete columns and reinforced concrete bearing walls at the stairs and elevator shaft. The concrete columns are typically 25'-0" on center in each direction. The foundation system of the building consists of a reinforced concrete mat footing.

Field investigation of the building indicated that the crack patterns observed in the top of the slabs at the columns and the bottom of the slabs at the center of bays were consistent with the expected crack patterns. Field testing with a magnetic rebar locator indicated that flexural reinforcing in the floor slabs appears to be as specified on the contract documents. Analysis has indicated that if flexural reinforcing is as specified on the contract drawings with a yield stress of 60 ksi as determined by previous testing, it is sufficient to support the specified design live load of 80 psf. Core testing of the concrete in the floor slabs indicated that the average concrete

strength is approximately 2,100 psi. The concrete strength specified on the contract documents is 3,000 psi. Petrographic analysis of the concrete determined that the cause of the low strength of the concrete was due to a high water-cement ratio in the concrete mix. Although many of the aggregates observed in the cores used for petrographic analysis are typical of those susceptible to alkali-silica reaction, there is little to no evidence of alkali-silica reaction products in the concrete.

The live load capacities of the floor slabs are highly variable and are controlled by the punching shear capacity of the floor slabs at the columns. The low concrete strength causes a reduction in the punching shear capacity. Live load capacities are lower in the upper levels of the building due to smaller column sizes at the upper levels at the decreased shear plane perimeter around the smaller columns.

Deflection analysis of the floor slabs was conducted to compare the theoretical calculated deflection with the actual deflections measured during the field investigation. Short-term deflection was calculated, then long-term deflection due to creep and shrinkage in the concrete was added to the short-term deflection to calculate the total deflection. The additional long-term deflection was calculated as approximately 2 times the short-term deflection. The calculated total deflection is consistent with the measured deflections. The average center bay deflection measured at the floors during the field investigation was approximately 2 1/4". Because creep and shrinkage in the concrete are almost totally complete within five years of the concrete being placed, almost all of the total deflection in the slabs would be complete within the first five years of service.

The seismic study of the building has indicated that in its present condition, the building lacks sufficient shear and bending capacity in the concrete shear walls in both of the major axes of the building when subjected to seismic zone 2B earthquake forces. The shear walls are highly overstressed in the east-west direction. In the north-south direction, the level of overstress in the shear walls is not as severe, but is still significant. It is recommended that new concrete shear walls be constructed to strengthen the building in both directions and provide increased life safety and lateral force capacity under zone 2B earthquake forces.

It is also recommended that new concrete column capitals or structural steel collars be constructed at columns with insufficient punching shear capacity. Because the

flexural capacity of the floor slabs appears to be sufficient to support the original design live load of 80 psf, it is recommended that the punching shear strengthening measures be designed and constructed to provide sufficient capacity to support the original design live load of 80 psf. Even if the punching shear strengthening measures are completed, live loads should be monitored closely to keep live loads within the capacity of the floor slabs.

Based on results of the field investigation and structural analysis, the deflections in the slabs are a serviceability problem and do not appear to be a strength or load capacity problem. If a reduction of the slab deflections is desired, the easiest and most economical method will most likely be to add a new leveling material or topping on top of the existing slabs. Due to the added dead load of the new topping, the live load capacity of the floor slabs will be decreased, but still should be greater than the required code minimum live load. Strengthening measures to increase the punching shear capacity must be completed prior to placing new topping on the existing slabs

STRENGTH EVALUATION AND STRUCTURAL ANALYSIS INVESTIGATION

1. PURPOSE AND SCOPE

This report provides an evaluation and analysis of the Albuquerque, New Mexico Federal Building at 517 Gold Ave. SW based on observations and analyses made during the months of August through December 1993. Site observations and measurements were performed at selected locations within the structure.

The purpose of this structural evaluation was to:

- a. Evaluate the existing floor slabs to determine their load carrying capacity
- b. Evaluate the existing floor slabs to determine the cause of the deflections of the floor slabs.
- c. Review the previous Structural Floor Analysis dated November 5, 1992, and compare the assumptions, results, and conclusions of the previous study with those of this study.
- d. Review the previous Seismic Study dated April 1992 and provide comments on the assumptions, methods, conclusions, and recommendations of the previous study.

The scope of the structural evaluation was performed in accordance with the "Scope of Work" prepared by the General Services Administration, dated February 9, 1993 and included the following:

- a. Study the existing contract drawings for the building.
- b. Perform on-site observations and measurements of existing structure. The on-site work included taking concrete core samples and performing Windsor Probe tests at each of the floors to determine the approximate strength of the existing concrete, observation of the top and bottom surfaces of the floor slabs at selected locations to document crack patterns in the floor slabs, magnetic rebar tests to verify to the extent possible the reinforcing steel in the slab, measurements of deflections in the existing floor slabs at various locations, and measurements of the existing in-floor electrical duct.
- c. Perform petrographic analysis of the concrete in the existing floor slabs to provide a qualitative analysis on the concrete.

- d. Perform a complete gravity load analysis of the existing floor slabs to determine the live load capacities of the floor slabs.
- e. Conduct research and analysis to calculate the theoretical deflections in the existing floor slabs, and compare the results of this analysis to the measured deflections in the floor slabs.
- f. Review the previous Structural Floor Analysis and compare the results and conclusions of the previous study with those of this study.
- g. Review the previous Seismic Study and provide comments on the assumptions, methods, conclusions, and recommendations of the previous study.
- h. Provide recommendations on strengthening measures for the existing floor slabs and lateral load resisting system.
- i. Provide preliminary construction cost estimates for recommended strengthening measures.

2. BUILDING DESCRIPTION

The Albuquerque Federal Building is an eight story concrete building with a full basement below grade. The building is rectangular in configuration and is 300 feet long by 102 feet wide. Floor to floor heights are 11'-0". Mechanical and elevator equipment penthouses exist on the roof of the building.

The structural system of the existing building consists of reinforced concrete flat slabs at the floors and roof. The flat slabs are 8" thick and are supported for the most part by reinforced concrete columns at 25'-0" on center in each direction. At a few locations the column spacing varies slightly from the typical 25'-0" spacing. Portions of the floor and roof slabs are supported by the reinforced concrete walls at the central elevator shaft and the east and west stairways. The floor and roof slabs are 300 feet long and 102 feet wide. The original contract drawings indicate that the flat slabs were designed with column and middle strips in both directions of the slab. The arrangement and dimensions on the column and middle strips meet the requirements of the current ACI 318 code. Reinforcing steel typically consists of #5 bars and is provided at the top and bottom of the slabs as required by bending in the slabs.

An electrical duct system has been cast in the floor slabs. The top of the electrical ducts is 2 1/2" to 3" below the top surface of the slabs, and is approximately 1"

deep. The width of the duct is approximately 11". Junction boxes exist where the electrical ducts cross. The contract drawings indicate the junction boxes are 1'-3 1/4" square. The covers at the junction boxes are approximately 10" square.

Drop heads exist at the floor and roof slabs at each of the columns supporting the slabs. The drop heads are typically 8'-4" x 8'-4" x 4" deep at the interior columns. At exterior and corner columns the drop heads extend 4'-2", or 1/6th of the span of the slab, from the centerline of the column in all directions where slab exists. The contract drawing indicate that shear head reinforcing has been placed in the slabs at each of the columns. This shear head reinforcing is placed in a form typically called "lampshade" reinforcing. A bent reinforcing bar is welded to a square ring of rebar with the plane of the bent bar perpendicular to the shear failure plane in the slab. See Figures 1 and 2 for a diagram of the "lampshade" reinforcing. At the edges of the slab at the exterior of the building, a perimeter spandrel beam exists, and provides support for the edge of the flat slabs as well as the exterior wall of the building. These exterior walls consist of unreinforced masonry with punched openings at the windows.

The foundation system of the building consists of a reinforced concrete mat footing according to the contract drawings. Reinforced concrete foundation walls exist at the perimeter of the building between the basement floor and the first floor. Interior partition walls are unreinforced masonry for the most part, but site investigation indicated that some partition walls consist of gypsum board on studs. The gypsum board partition walls are probably the result of remodeling of the interior spaces.

The lateral force resisting system of the building consists of reinforced concrete shear walls at the elevator shaft and stairways. The concrete shear walls also support floor loads as mentioned earlier, so the lateral force resisting system must be considered to be a bearing wall system. Many of the concrete shear walls do not have sufficient strength to resist lateral earthquake forces specified by the current Uniform Building Code as discussed in section 6.

3. FIELD INVESTIGATION

The first step of the field investigation was to remove floor finishes and ceilings so the crack patterns in the slabs could be viewed and documented. Removal of the finishes was done by a contractor employed by the building manager. Floor finishes were removed over an approximate 12'-0" x 12'-0" area to expose the top surface of the floor slabs at grids F & 4 at level 8, grids E & 4 at level 7, grids E & 2 at level 6, and grids K & 4 at level 5. The ceiling was removed at one bay in the fourth floor conference room to expose the bottom surface of the level 5 floor slab between grids 3 & 4 and grids J & K. Once the floor and ceiling finishes were removed, a visit to the site was made to view the crack patterns and complete other field investigation.

The crack patterns observed in the top surfaces of the slabs were configured in a "star" pattern with the cracks radiating out from the columns. This crack pattern is shown on Figure 3 and is consistent with the expected crack pattern in the top of the slab. Observed crack patterns in the top surfaces of the slabs are shown on Photos 1 through 15. Crack sizes were measured with a crack comparator. Crack sizes in the top surfaces of the slabs varied from hairline cracks approximately .003 inches wide, to a maximum width of approximately .035 inches. Near the columns the cracks were wider and usually in the .020 to .030 inch range. The cracks decreased in width with increasing distance from the columns. At distances of 5 to 6 feet from the columns, the width of the cracks varied from hairline cracks to approximately .016 inches wide. Crack spacing was typically 6" to 12" on center. The numbers written on the slab next to the cracks in the photos are the width of the cracks in inches.

The crack pattern observed at the bottom surface of the level 5 floor slab was confined to areas of bending tension in the slab, and was consistent with the expected crack pattern shown in Figure 4. The crack widths did not exceed a hairline crack with approximate width of .003 inches. In many areas of the slab it was difficult to see the cracks due to the small crack width and the uneven plywood formed surface on the bottom of the slab. Crack spacing was typically 6" to 12" on center as shown on Photos 15 through 18.

In conjunction with the investigation of the cracks in the floor slab, magnetic rebar tests were performed to verify to the extent possible, the existing rebar in the floor slabs matched that shown on the contract drawings. A magnetic rebar locator was

used for this rebar verification. The rebar locator is basically a metal detector. It has a gauge with a needle which will show increased magnetic field when the rebar locator passes over a reinforcing bar. If the concrete cover over the bar is known, the rebar locator will give a fairly close indication of the bar size at that concrete cover, or if the bar size is known, the rebar locator will give a fairly close indication of the concrete cover over the bar.

Rebar verification using the rebar locator was performed at the top surface of the floor slabs at grids F & 4 at level 8, grids E & 4 at level 7, grids E & 2 at level 6, grids K & 4 at level 5, grids K & 2, E & 4 and D & 3 at level 4, and grids K & 2 and F & 3 at level 3. Readings on the rebar locator at these locations appeared to be consistent with #5 bars with 3/4" concrete cover to the top layer as specified for the rebar on the contract drawings. Rebar locations were determined for the bars in both directions, and marked on the exposed slab, or where floor finishes were not removed, a pencil or other object was placed on the floor to indicate the bar location. Some of the rebar location marks on the slabs can be seen in the photos showing crack patterns. The spacing of the bars was then measured and compared with the bar spacing calculated using the quantity of reinforcing bars indicated on the contract drawings spaced evenly over the width of the column strip.

Rebar verification was performed at the bottom surface of the level 5 floor slab where the ceiling was removed in the level 4 conference room. Readings from the rebar locator were consistent with a #5 rebar with 1" concrete cover to the lower layer as specified for the rebar on the contract drawings. Rebar locations were determined using the same procedure used for the top surfaces as discussed above. The spacing of the existing bars was then compared to that calculated using the quantity of reinforcing bars indicated on the contract drawings spaced evenly over the width of the middle strip. Based on the results of the rebar investigation, it appears that the existing rebar in the slabs was placed according to the contract drawings at the ten locations tested.

After the rebar verification was completed, concrete cores were taken for strength tests and Windsor Probe tests conducted to determine the strength of the existing concrete in the floor slabs. Two concrete core samples were taken, and six Windsor Probe tests were conducted at each floor slab except at level 1 where only one core sample was taken due to the lack of water at one end of the building, and level 5

where only five Windsor Probe tests were conducted. One of the core samples from level 6 was used to perform petrographic analysis, so only one of the cores from level 6 was broken for a concrete strength test. The core samples and Windsor Probe tests were taken at either the areas where the top surface of the slab was exposed, the floor at the east mechanical room between grids L and M, or the floor at the west mechanical room between grids B and C. The location of the mechanical rooms was chosen because there is no floor finish at these rooms other than paint applied directly to the concrete floor slab.

All core sampling and testing was performed by SHB AGRA Inc. of Albuquerque. The results of the concrete strength tests are shown in Appendix A-2. The strength of the concrete cores varied from a low of 1810 psi to a high of 2570 psi. The average strength of the 14 cores that were broken was 2176 psi. The standard deviation was 277 psi. The average strength of the concrete based on the results of the Windsor Probe tests was 2178 psi which is very close to the average strength of the concrete cores. The contract drawings indicate that the concrete in the floor slabs should have a minimum compressive strength of 3,000 psi. The actual strength of the concrete is significantly lower than this.

The field investigation also included measuring the existing deflections in the concrete floors at various locations in the building. Deflections were measured at levels 1,3,4,5,6,7 and 8 using an automatic level and a fiberglass level rod. The slab deflections at the locations measured are shown in Figures 5 through 11. The deflections shown are with respect to the top of slab elevation at the columns, and are expressed in decimal feet. For example, .17 indicates a deflection of 17/100's of a foot or approximately 2 1/16 inches. At many locations the top of slab elevations at adjacent columns varied slightly. Where this elevation variation existed, the listed deflections are with respect to the average elevation at the columns. Deflections at the slab between columns are with respect to the two columns on each end of the span. Deflections at the center of the bay are with respect to the average elevation of the four columns at the corners of the bay.

Typical deflections measured at the center of the bays varied from 0.16 to 0.22 feet or approximately 1 15/16 to 2 5/8 inches. The average measured deflection at the center of the bays was 0.186 feet or approximately 2 1/4 inches. The deflections measured at the center of the bays adjacent to the elevator shaft were somewhat less

and varied from 0.12 to 0.15 feet or approximately 1 7/16 to 1 13/16 inches. The decreased deflections at these bays are due to the support provided by the elevator shaft concrete walls and lack of significant deflection along one edge of the bays. Typical deflections measured at the slab between columns varied from 0.08 to 0.16 feet or approximately 1 to 1 15/16 inches. Where support is provided by walls or beams, the measured deflections were lower than this. Considering the many variables which contribute to deflection on the floor slabs, the measured deflections are quite consistent.

Verification of the in-floor electrical duct was possible at only two or three locations where the junction boxes were accessible, and indicated that the duct and junction boxes were placed as described earlier in the building description section.

4. EVALUATION AND FINDINGS

Floor Load Capacity

The floor live load capacities were determined using the ADOSS computer program developed by the Portland Cement Association. The ADOSS program is a design and analysis program for concrete flat slab structures. The program is based on the equivalent frame analysis method of ACI 318. The program analyzes the slab under load cases with all spans loaded as well as load cases with alternate spans and two adjacent spans loaded. A concrete strength of 2,100 psi was used for the analysis which is slightly less than the average of the concrete core strength tests and Windsor Probe tests. A yield stress of 60 ksi was used for the reinforcing bars based on tests performed on a rebar sample taken from the sixth floor slab in 1992 by Geo-Test of Albuquerque. The sample tested had a yield strength of 77.3 ksi and an ultimate strength of 87.0 ksi. The original contract specifications do not clearly define the required rebar yield stress, and specify "bars QQ-B-71a, type B, conforming to ASTM A305-50T and any grade except structural grade". Flexural reinforcing in the floor slabs was assumed to be as shown on the contract drawings based on the results of the field investigation.

The "lampshade" shear head reinforcing was neglected in the analysis of the floor slabs. This type of shear head reinforcing is no longer allowable under the ACI 318 code and its ability to develop the required force in the inclined bar has been

seriously questioned. The plan dimension of the shear head reinforcing is constant at 2'-2" at the top of the inclined bars and 3'-11" at the bottom of the inclined bars as shown on Figures 1 and 2. Because the column size varies, the location of the shear failure plane varies with respect to the inclined shear bars of the shear head reinforcing. At the smaller 12" x 12" and 14" x 14" columns in the upper levels of the building, the shear plane intersects the inclined bars very near the top of the inclined bars as shown on Figure 2. This leaves very little, if any, embedment length for the inclined bars on one side of the shear plane. The inclined bars across the shear plane cannot be developed due to this short embedment length. The shear head reinforcing will be almost totally ineffective in these cases. Where the columns are larger and the length of embedment on each side of the shear failure plane is more even, the shear head reinforcing will be more effective, but the reinforcing still cannot be considered to be fully effective. For this reason the shear head reinforcing has been neglected in the gravity load analysis of the floor slabs.

The gravity analysis of the floor slabs using the ADOSS program and the above criteria has indicated the live load capacities of the floor slabs are limited in most cases by punching shear capacity at the columns and not by the flexural slab reinforcing. At the upper levels of the building where the columns are smaller, the smaller perimeter dimension of the shear failure plane around the columns, coupled with the low strength of the concrete, limits the punching shear capacity and thus limits the live load capacity of the floor slabs. At the lower levels of the building, the columns are larger, so the perimeter dimension of the shear failure plane around the columns is increased and the live load capacity increases. In all cases the punching shear capacity of the floor slabs is lower than the punching shear capacity under the original design of the floor slabs. This is due to the actual average concrete strength of 2100 psi being lower than the design concrete strength of 3,000 psi.

The live load capacities of the floor slabs at each level are shown on Figures 12 through 17. The weight of interior partition loads must be included in the live load capacities shown on these figures. The analysis of the floor slabs assumed the partition weight would be included in the live load due to the variability of the partition weight. In some areas of the building there are no partitions other than those included as part of the modular furnishings. At these areas the full live load capacity of the floor slab can be used for occupants, furnishings, contents, etc.. At other areas where interior partitions exist, the weight of the partition walls must be

deducted from the live load capacity shown on the figures to determine the allowable weight of furnishings, contents, occupants, etc. that may be placed on the slab. The partition weight in the building is quite variable, but for the most part, appears to be in the range of 10 to 20 psf.

The main reason for the variability in the live load capacity shown on the figures is the variation in the column sizes at each level. Where columns are larger, the live load capacity will increase. The live load capacity at the exterior bays is less than the interior bays due to the distribution of the slab shears in the exterior span between the exterior column and first interior column. The slab shear is not distributed evenly between the exterior column and first interior column. The slab shear is greater at the first interior column and is based on the equivalent frame analysis. Because of the increased shear at the first interior column, the live load capacity at the exterior slab spans is less than at interior spans where the shear distribution between columns is more even.

Another reason for the decreased live load capacity at the exterior bays is that the ADOSS program calculates the effect of unbalanced moments on the punching shear at the columns and includes this in the analysis. Unbalanced moments will cause an increase in the punching shear stress on one side of the column and a decrease in the punching shear stress on the other side of the column. Unbalanced moments are greater at the exterior columns and first interior columns than at the interior columns in most cases. The live load capacities shown on the figures are calculated so that the punching shear stress combined with the shear stress due to unbalanced moments does not exceed the allowable shear stress under the controlling load case. Shear calculations for each of the frame lines are shown in the shear analysis section of the computer output submitted with this report.

Tables 1 through 4 are a comparison of the allowable slab moments compared to the required slab moments at two of the more typical frame lines at levels 2 through 8. Required bending moments are based on an 80 psf live load per the contract drawings, and are taken from the equivalent frame analysis of the ADOSS computer program. Allowable bending moments are based on the 2,100 psi concrete strength used in the shear analysis with 60 ksi rebar. The bar size, quantity, and configuration shown on the contract drawings was used to calculate the allowable bending moments. The allowable bending moments meet or exceed the required bending

moments in all cases considered. Flexural strength is not as sensitive to variations in concrete strength as punching shear strength. This is evident in the results of the analysis which indicate the floor live load capacities are limited by punching shear capacity. Flexural analysis of all frame lines at levels 8 and 5 using the ADOSS program are included with the calculations submitted with this report. Comparison of the required flexural reinforcing from the computer output, to the reinforcing provided in the slabs indicate that the flexural reinforcing is sufficient to support the specified 80 psf live load.

Subsequent to the analysis of the floor slabs, computer analysis was performed for two of the more typical frame lines at the roof slab. Frame lines at grids D and 2 were analyzed. No testing or field investigation was performed on the roof slab, so the same criteria used to analyze the floor slabs was used in the roof slab analysis. The reinforcing and slab information shown on the contract drawings was also used in the roof slab analysis. The analysis indicates that the roof live load capacity at these two frame lines meets or exceeds the 20 psf live load capacity specified on the contract drawings.

The interior columns on grids D, E, and F at grid 3, which represent some of the more typical columns in the building, were analyzed to check the capacity of the columns using a reduced compressive strength. The contract documents indicate that the columns were originally designed with f'_c equal to 3,750 psi. The columns were analyzed with f'_c equal to 2,100 psi to match the strength used in the slab analysis, and f'_c equal to 2,600 psi. The f'_c of 2,600 is 70% of the design strength to match the proportion between the actual strength of 2,100 psi compared to the design strength of 3,000 psi for the concrete in the floor slabs. Testing of the concrete in the columns is beyond the scope of this evaluation and was not included in our field investigation. The columns were analyzed using live load reductions allowed by chapter 23 of the Uniform Building Code.

The column analysis indicated that for a concrete strength of 2,600 psi, the columns appear to have sufficient capacity to support the design live load of 80 psf on the floor slabs. For a concrete strength of 2,100 psi, the analysis indicated that the column capacities are very close to the required column capacity under an 80 psf floor live load. Some columns appear to have a capacity slightly less than required under an 80 psf floor live load according to the analysis using an f'_c of 2,100 psi. It

is possible that columns at other grids may be overstressed under an 80 psf floor live load using an f'_c of 2,100 psi. If a higher level of confidence in the column capacities is desired, further analysis of the concrete columns should be performed to determine to the extent possible, the actual concrete strength of the concrete in the columns and the associated capacity of the columns.

Petrographic Examination

Petrographic examination of concrete samples from the floor slabs was performed to try to determine the reason for the low strength of the concrete. Initially two core samples were submitted to Hi Tech Consulting of Salt Lake City, Utah for petrographic analysis. A copy of the petrographic examination report is included in Appendix A-2. One core was from the 6th floor slab and one core was from the 1st floor slab. Results of this petrographic analysis were somewhat inconclusive, and suggested that the reason for the low strength of the concrete may be due to carbonation of the concrete matrix as well as possible alkali-silica reaction in the concrete. Alkali-silica reaction is a reaction between silica in certain types of aggregates with alkali in the cement paste of the concrete. The petrographic analysis indicated that some of the aggregates were potentially reactive. The reaction between the alkali and silica produces an alkali-silica gel that can weaken the concrete when it is exposed to moisture.

Alkali-silica reaction in concrete is very serious and its weakening effects cannot be reversed. For this reason, a second opinion on the cause of the low concrete strength was sought. Construction Technology Laboratories, Inc. (CTL) of Skokie, Illinois was retained to perform further petrographic examination and testing on the concrete. CTL is widely known as one of the best testing labs and authorities on concrete in the nation. Four additional core samples for petrographic examination were taken from levels 3,4,7, and 8. Three additional cores samples for concrete strength tests were taken from levels 4,7, and 8. As part of the concrete strength tests, modulus of elasticity tests were performed on the cores from levels 4 and 7. A copy of the petrographic examination report by CTL is included in Appendix A-2.

Petrographic examination by CTL indicated that while many of the aggregates observed in the cores are typical of those susceptible to alkali-silica reaction, there is little to no evidence of reaction products in the concrete. Test results indicated that the poor quality of the concrete was primarily due to a high water-cement ratio of

over 0.65. The depth of carbonation from the top surface of the slab was variable with a maximum depth of 3/4". The effects of this carbonation on the strength of the concrete are insignificant. The results of the modulus of elasticity tests indicate that the concrete modulus of elasticity is quite low with an average value for the two tests of 2,050,000 psi. The average compressive strength for the three strength tests was just under 1,800 psi which is less than the 2,100 psi compressive strength used in the floor analysis. This lower strength is most likely due to the small sample of only three cores used in the tests. If the three additional concrete strength tests conducted by CTL are averaged into the compressive strengths from the original testing program, the average value is still greater than the 2,100 psi average compressive strength used in the slab analysis.

Deflection Analysis

Deflection analysis of the floor slabs was conducted to compare the theoretical calculated deflections with the actual deflections measured during the field investigation. The initial deflection calculations were performed by the ADOSS program during the gravity load analysis of the floor slabs. The ADOSS program calculates short term deflections based on the procedures of ACI 318 which have been adapted to use for flat slabs by an ACI Journal article in December, 1976 titled "Short-Time Deflection of Flat Plates, Flat Slabs, and Two-Way Slabs". This method of deflection calculation is compatible with the equivalent frame method of analysis and uses the effective moment of inertia of the slab section under consideration to calculate short-term deflections. The effect of cracking due to bending forces in slabs is to reduce the flexural stiffness of the slab section under consideration and thus increase deflections in the slab over the deflections that would be calculated using the gross section moment of inertia for an uncracked section. The effective moment of inertia will be greater than the cracked moment of inertia calculated for the slab section due to the stiffening influence of the tensile concrete between cracks in the slab section and due to the variability of the cracking over a bay width or length. The calculated short-term deflection will thus be somewhere in between the deflections that would be calculated using the gross section moment of inertia and the cracked section moment of inertia.

Additional long-term deflections resulting from creep and shrinkage in the concrete were calculated using the methods of section 9.5.2.5 of ACI 318-89. The additional long-term deflection calculated using the equation in this section using an area of

compressive reinforcing of zero (which is the case in most locations of a flat slab), is 2 times the short-term deflections for a time greater than 5 years. This results in total deflections 3 times the short-term deflections. Research published in Special Publication 43-3 by ACI Committee 435 suggests that the additional long-term deflections for singly reinforced members such as flat slab structures will be closer to 2.5 times the short-term deflections.

Table 5 shows the short-term deflections calculated by the ADOSS program, the additional long-term deflections based on the long-term deflections being 2 times the short-term deflections, and the total deflections. As can be seen from the table, the calculated total deflections are within the range of variation of the deflections measured during the field investigation. If additional long-term deflections are calculated as 2.5 times the short-term deflections, the total deflections will increase slightly (about 17%), and will still be within the range of variation of the deflections measured during the field investigation.

Additional long-term deflections were also calculated using the procedures of an article titled "Prediction of Long-Term Deflection of Flat Plates and Slabs" in the April 1976 ACI Journal and the information contained in ACI 209 "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures". Using the criteria from these publications, the additional long-term deflection due to creep was calculated to be 1.44 times the short-term deflection. The additional long-term deflection due to shrinkage was calculated to be 0.3 inches for the 25 foot slab span. Table 6 shows the short-term, long-term, and total deflections based on this method of calculation. The total deflections are close to those calculated using the other methods considered and are within the range of variation of the deflections measured during the field investigation.

Finite element analysis was performed for different portions of the concrete floor slab in order to check the short-term deflections calculated by the ADOSS program. The conditions modeled using finite element analysis included an interior bay, exterior bay, corner bay, and four bay condition with two interior bays and two exterior bays. Support conditions for continuous slab edges were modeled to allow vertical translation of the slab edge, but restrained rotation of the slab edges. The exterior edges of the concrete floor slab were restrained against vertical rotation for the exterior bay and corner bay conditions. The existing spandrel beams at the exterior

edges of these bays will allow only limited vertical deflection of the concrete floor slab, so the vertical translation was restrained at these edges. The slab was analyzed using the finite element analysis with the exterior slab edges restrained against rotation as well as unrestrained against rotation. The torsional stiffness of the concrete spandrel beam will provide some restraint against rotation, but the slab edge will not be fully restrained. The deflections at the center of the bay for the restrained rotation case and the unrestrained rotation case were averaged to obtain the theoretical deflection at the center of the bay. The slab deflections calculated by the finite element analysis are for a loading that includes dead loads plus a superimposed live load of 80 psf. The modulus of elasticity used in the finite element analysis was the 2,050,000 psi average value from the modulus of elasticity tests performed by CTL.

The thickness of the concrete slab was modeled in the finite element analysis to be consistent with the average effective moment of inertia of the slab. If the full 8" thickness of the concrete slab were used in the computer model, the deflections would be based on the gross moment of inertia of the uncracked section, and the calculated deflections would not be reflective of the real conditions in the floor slab. The actual negative bending moments in the floor slabs are approximately 4 times the cracked moment at most frame lines. The actual positive bending moments in the floor slabs are approximately 1.8 to 2 times the cracked moment at most frame lines. The high bending moments are evident in the cracks observed in the floor slabs during the field investigation. The moment of inertia must be modified to reflect the actual conditions in the slab.

The effective moment of inertia was calculated for the slab at the columns for negative bending and the middle of the spans for positive bending for the more typical frames in each direction of the floor slab. These effective moments of inertia are then used to calculate the average effective moment of inertia for the frame in each direction. The average effective moments of inertia for the frames were then averaged to calculate an average effective moment of inertia for the entire bay of the floor slab. This average effective moment of inertia was used to calculate a slab thickness that would yield a gross uncracked moment of inertia equivalent to the average effective moment of inertia. A slab thickness of 6" has a gross moment of inertia equivalent to the average effective moment of inertia in the slab and was used in the finite element analysis. The short-term deflections calculated by the finite element analysis are shown in Tables 7 and 8 and are close to those calculated by the

ADOSS program. Plots of the slab deflections from the finite element analysis are shown on Figures 18 through 23. When additional long-term deflections are added to the short-term deflections calculated by the finite element analysis, the total deflections are consistent with the other methods of deflections calculation and are within the range of variation of the deflections measured during the field investigation and very close to the average measured deflection of 2 1/4 inches at the center of the bay.

As can be seen from the deflection tables, the deflections total under all methods of calculation are consistent with the deflections measured in the field. Both methods used to calculate additional long-term deflections are consistent with one another as well as the deflections measured in the field. The apparent cause of the total deflections observed in the floor slabs is due to the summation of the initial short-term deflections and the long term deflections due to effects of creep and shrinkage. Because creep and shrinkage in the concrete are almost totally complete within five years of the time the concrete is placed, almost all of the slab deflections would have occurred during the first five years of service. The in-floor electrical ducts have negligible effect on the slab deflections, if any at all.

5. REVIEW OF STRUCTURAL FLOOR ANALYSIS

The previous structural floor analysis dated November 5, 1992 by BPLW Architects and Engineers was reviewed to compare the assumptions, calculations, test results, and conclusions of this evaluation and analysis with the analysis performed by BPLW. The results of our concrete strength testing program are quite close to the results of the BPLW testing program. The average strength of core samples taken during our initial and secondary sampling and testing was just over 2,100 psi. The average strength of the three core samples under the BPLW testing program was 2,283 psi and is within the range of strength values obtained under our testing program. The variation in the average values under the BPLW testing may be due to the samples being taken from only one floor rather than spread throughout the building. The small number of samples may also be a reason for the variation.

Results of the equivalent frame analysis of the flat slab floor structures performed by the ADOSS program under this study are fairly close to those of the BPLW study in

many areas, but the allowable live load due to limitations of punching shear capacity are lower in many areas under our analysis. This difference in live load capacity appears to be due to the following factors. The lower concrete strength which resulted from our testing program, and was used in the analysis of the floor slabs, is one reason the live load capacities due to punching shear are less. Another reason for the reduced live load capacity due to punching shear, especially at the exterior bays, appears to be due to inclusion of the contribution of unbalanced moments to the punching shear stress by the ADOSS program analysis. The uneven shear distribution between the exterior column and first interior column at the exterior slab spans also appears to be a reason for the lower live load capacities. The punching shear calculations shown for level 6 in Appendix B of the BPLW report do not appear to take into account the effects of unbalanced moments and the uneven distribution of shear in the exterior spans. Punching shear calculations for floors other than level 6 do not appear to be included in the report. A check of the allowable live load at level 8 using the more typical 14" x 14" column size, the 2,300 psi concrete strength, and the same calculation procedures as the BPLW report, indicate an allowable live load of 35 to 40 psf rather than the 53 psf live load capacity shown in the table at the front of Appendix B of the BPLW report. The reason for this discrepancy is not known due to the apparent lack of punching shear calculations in the report for level 8.

The short-term deflection calculated by finite element analysis which is shown in Appendix C of the BPLW report is quite close to the deflections calculated by our analysis using the ADOSS program and finite element analysis. The small difference in the calculated short-term deflection is most likely attributable to the lower modulus of elasticity used in our analysis. The short-term deflection calculation by BPLW used a modulus of elasticity of 2,734,000 psi based on a concrete strength of 2,300 psi and was calculated using the methods of ACI 318. Our short-term deflection calculations were based on a modulus of elasticity of 2,050,000 psi which was determined as part of the testing performed by CTL.

The total short-term and long-term deflections calculated by our study differ significantly from those in the BPLW report. This is due to differences in the calculation of additional long-term deflections. The calculated short-term deflections shown in Appendix C of the BPLW report for the bay considered are shown as 0.649 inches. The calculations show an amplification factor for creep of 2.0. Total deflection is then calculated as 2 times the short-term deflection or 1.297 inches. A

totally cracked section is then assumed and the total deflection is proportioned up to 1.67 inches. This calculation sets the additional long-term deflection (in this case due to creep) equal to the short-term deflection. No mention, or calculation of additional deflection due to concrete shrinkage is apparent in the calculation.

It appears that the calculations have attempted to follow the procedures of section 9.5.2.5 of ACI 318, but have not properly calculated the full magnitude of the additional long-term deflection. The equation in this section is used to calculate "additional long-term deflection resulting from creep and shrinkage of flexural members". The additional long-term deflection is calculated by multiplying the short-term deflection by the factor from the equation (2.0 for a singly reinforced member with time greater than 5 years). The additional long-term deflection is then added to the short-term deflection to obtain the total deflection. Using the long-term factor of 2.0, the total deflection will be 3 times the short-term deflection. If the short-term deflection of 0.649 inches for the condition with the slab edges restrained in the BPLW report is multiplied by three, a total deflection of 1.947 inches or .162 feet results at the middle of the bay. This is quite close to the average mid bay deflection of 2 1/4 inches or .186 feet measured during our field investigation.

The recommendations section of the BPLW states that "the extreme deflections measured constitute visible evidence of failure", but later in the same section, questions whether a partial failure has actually occurred. Our analysis has indicated that the measured floor deflections are consistent with the total deflections that are to be expected using current techniques to calculate total deflections. It is our opinion that the deflections do not constitute visible evidence of failure, and do not indicate partial failure. The occurrence of overload, bond failure, improper shoring, and plastic deflection in the structure are possible, but not probable. Field investigation appears to indicate that the flexural reinforcing was placed according to the contract drawings. Analysis has indicated that the flexural reinforcing shown on the contract drawings is sufficient to support the design live loads. In our opinion the deflections are a serviceability problem and do not indicate flexural failure of the slab system. The live load capacities of the floor slabs are limited by punching shear capacity. The low punching shear capacity does not contribute significantly to the slab deflections.

6. REVIEW OF SEISMIC STUDY

The previous seismic study dated April, 1992 by BPLW Architects and Engineers was reviewed and limited analysis performed to verify where required, the assumptions, methods, conclusions, and recommendations contained in the BPLW report. The lateral analysis by BPLW has used an R_w factor of 8 in the calculation of the seismic forces on the building. This R_w factor indicates that the analysis was performed for concrete shear walls with an essentially complete gravity frame system. It is our opinion that the building should be analyzed as a concrete bearing wall system with the lower R_w of 6. Walls at both the elevator shaft and the stairways support floor slabs and concrete beams. This is especially true at the west wall of the elevator shaft. The lower R_w factor will cause an increase in the calculated base shear for the building.

The ETABS program for three dimensional analysis of building systems was used to analyze the shear walls that constitute the lateral force resisting system of the building. For our lateral analysis, the base of the building was considered to be at the level 1 floor slab due the presence of the concrete foundation walls around the perimeter of the building between the basement floor and level 1. A static analysis was performed on the building. The fundamental period of vibration was calculated for the lateral force resisting system in each of the principle axes of the building by ETABS. The period in the north-south directions was calculated as 0.85 seconds. The period in the east-west direction was calculated as 1.98 seconds. The period calculated using method A in chapter 23 of the Uniform Building Code was 0.575 seconds. The calculation of the C factor is limited to a minimum of 80% of the C factor calculated using the period from method A. This yields a C factor of 1.735 which was used in the analysis. Using the above factors and $Z=0.2$, and $I=1.0$, the base shear was calculated to be 2713 kips. The base shear used in the BPLW analysis is 2594 kips with the base of the building considered to be at the basement floor.

The ETABS analysis indicates that the existing concrete shear walls are highly overstressed in shear and overturning in the east-west direction. The analysis also indicates that some of the shear walls in the north-south directions are overstressed in shear at some locations. Tables 9 through 11 show the required shear compared to the allowable shear in the major walls in the north-south direction. The major shear walls in the north- south direction were analyzed for overturning and a number of the

walls were found to be highly overstressed under overturning forces. The walls with high overstress in overturning are the west wall of the elevator shaft, the east wall of the west stair, and the west wall of the east stair. The wall panels and lintels between door openings in the elevator shaft were not checked for overturning forces, but these are most likely overstressed in overturning also. The light vertical reinforcing in these walls and the lack of boundary elements or bending reinforcing is the reason for the overturning problems in these walls. Overturning forces due to the calculated seismic forces produce a net tensile bending force in the ends of the walls and there is not enough vertical reinforcing steel to resist the required bending force. The BPLW report does not appear to address overturning of the shear walls as required by section 2334(g) of the Uniform Building Code.

Because most of the existing shear walls are overstressed in shear and/or bending in both directions, it is our opinion that the building should be strengthened in both directions, and not just in the east-west direction. The exterior shear wall method of upgrade appears to be the most economical and least disruptive method to accomplish this strengthening. The new shear walls must be designed to either resist all of the lateral force in the building, or limit the forces in the existing shear walls to a level that will not produce shear or overturning overstresses in the existing shear walls. With new shear walls placed in both directions, the cost of the shear wall strengthening will be approximately double the cost shown under option 1A of the BPLW report.

The report by BPLW states that there will be no significant savings if 80% of the code lateral forces are used in the design of the seismic upgrade. If the new shear wall can be stepped so the walls are longer at the bottom of the building and shorter at the top of the building, the savings could be quite significant due to the decreased area of wall forming, concrete quantity, and demolition required. Stepped shear walls may not be possible due to aesthetic considerations, but even if the walls remain as shown under option 1A of the BPLW report, the reduced lateral forces will produce less overturning in the walls and there should be some fairly significant savings in the new foundation systems at the new shear walls.

Options 2A and 2B of the BPLW report use new steel moment frames to strengthen the existing building. The calculations do not appear to consider the interaction between the existing concrete shear walls and the new steel moment frames under

code mandated lateral forces. The new steel moment frames are quite flexible, especially under option 2B. Some of the interstory drift ratios calculated in the BPLW report for option 2B appear to exceed those allowed under section 2334(h) of the UBC. Distribution of the lateral force must be done according to the stiffness of the members. The new moment frames must be stiff enough to limit the shear and overturning forces in the existing shear walls to allowable levels. It is likely that partial or total failure of the existing shear walls will occur before the building will drift laterally to the point where the new steel moment frames will effectively resist the seismic forces. Because the shear walls also function as bearing walls at many locations, failure of the shear walls could result in collapse of the floor and roof slabs in some areas.

Because the existing shear walls are overstressed in both directions, new frames would be required to be placed in both directions. The shear walls in the north-south direction are much stiffer than the shear walls in the east-west direction. The new frames in the north-south direction must be designed considering the frame/shear wall interaction and must be stiff enough limit forces in the existing shear walls to allowable levels.

The lateral force calculations for options 2A and 2B appear to use an R_w factor of 8. This factor is not consistent with the values for moment frame or dual systems in Table 23-O of the UBC. An R_w factor of either 6 for a steel ordinary moment frame or 12 for a steel special moment frame system should be used. The same factors apply to dual systems. It is questionable whether the system can be classified as either a special or ordinary moment frame system. The new moment frames would need to be very stiff to limit the forces in the existing shear walls to a level where the forces were insignificant and the new moment frames resisted nearly all lateral load. It will most likely be unfeasible to add new moment frames with this high stiffness to the building. If the new moment frames are used to strengthen the building, the lateral system will most likely function more like a dual system due to the stiffness of the shear walls. Section 2333(f).5. of the UBC requires that dual systems have an essentially complete space frame which provides support for gravity loads. The existing shear walls also function as bearing walls so this requirement is not met. While the UBC technically does not apply to the upgrade of existing buildings, the intent of this requirement should be followed in the design of the seismic upgrade. Great care should be exercised in choosing an appropriate R_w factor, and insuring

proper behavior in the new moment frames and existing shear walls during a seismic event.

The frame elevations for options 2A and 2B show beams in the moment frames at every other floor instead of every floor. For option 2B, the computer model appears to place a beam at each floor, but the frame elevation shows a beam only at every other floor. The frame elevation should be reflective of the computer model or vice versa in order to provide an accurate analysis of the moment frame forces and behavior. It appears that these two options rely on the columns to resist the lateral forces at the floors where beams do not exist. The lateral forces at these floors impose a horizontal load at mid-height of the columns which creates a bending force in the columns in addition to the bending forces created by frame action. The load is imposed at the theoretical moment inflection point in the columns. The additional horizontal force at the columns will increase bending in the columns and shift the location of the inflection point. Care must be taken to insure that lateral drifts from floor to floor do not exceed the code allowable drift ratios. The additional lateral drift contributed by the column deflection which is produced by the mid-height column loads must be considered.

Another very important consideration in the analysis of the moment frames is maintaining stability in the columns when the frames are subjected to code level earthquake forces at the mid-height of the columns as well as at frame joints. The lateral earthquake forces calculated under the procedures of the UBC are significantly less than the actual lateral forces produced by an earthquake with lateral accelerations equivalent to the "code" level earthquake. The UBC and other codes make a basic assumption that the lateral force resisting system will be stressed beyond the elastic range and will behave in the post-elastic or plastic range. Great care must be taken to insure the columns are strong enough to maintain stability when subjected to the additional bending produced by the actual lateral seismic forces at mid-height of the columns rather than the calculated code level forces.

Non-structural Elements

The masonry screen wall at the roof appears to be about twelve feet high or about half the distance to the edge of the roof. It is possible that segments of this wall could be projected over the edge of the roof during a moderate to large seismic event.

We recommend that this wall be braced at mid height to increase the stability of the wall and reduce the life safety risk to pedestrians below.

We concur that the support beams for the cooling tower appear to be in need of an upgrade. The seismic adequacy of the anchorage for the cooling tower and other roof mounted equipment should also be considered.

There are suspended plaster ceilings throughout the building. It does not appear that these ceilings are braced to the structure. Most ceilings through the building, with the exception of the sixth floor, also support suspended acoustical lay-in ceilings and light fixtures. The UBC code does state in footnote No. 7 of Table No. 23-P that "suspended members that support a ceiling at one level extending from wall to wall need not be analyzed provided the walls are not over 50 feet apart". In most cases the suspended members appear to support two levels of ceilings, the plaster ceiling and the acoustical ceiling. The original walls do not appear to be braced to the structure above and would not qualify for the walls specified in footnote No. 7, Table 23-P.

Section 4703 of the UBC requires that plaster and gypsum board walls (vertical assemblies) be designed to resist seismic forces. This would require the walls to be braced to the structure. The only walls that appear to be braced or built into the structure above are at the stairways, the elevator shafts and possibly at the mechanical rooms. The other walls which were observed were either moveable partition walls, partition walls built under the acoustical tile ceiling, or partition walls that appeared to be discontinuous above the suspended plaster ceiling line. It did not appear that any of these partition walls were braced to the structure above. Even if the corridor walls were braced, the dimension in the opposite direction would exceed the 50 feet specified in footnote No. 7 and the ceiling would still require bracing.

It is recommended that the following items be braced to the structure as specified in chapter 23 of the UBC:

1. Suspended ceilings throughout the building.
2. Corridor walls and other partition walls.
3. Lay-in light fixtures throughout the building.

The cost estimates in the BPLW report appear to be quite detailed and accurate for the systems considered in the report. The cost estimates consider strengthening the building in only the east-west direction. Strengthening the building in both directions will approximately double the cost of the upgrade for the options considered. The cost to remove and replace heating convectors is included in the cost estimate for option 1A, but does not appear to be included in any of the other options. Will this removal and replacement be required under the other options, and if it is, has it been included in the cost estimates for the other options? The cost to brace the existing ceilings, partition walls, and light fixtures will add approximately \$300,000 to the seismic upgrade of the building. This figure is based on all of the existing ceilings, partitions, etc. remaining in place, and will decrease if all or portions of the existing finishes are removed and replaced to allow other construction. It does not appear that any contingency funds have been included in the cost estimate by BPLW. We would suggest that some contingency funds be added to the estimate to cover uncertainty in the estimate and unforeseen conditions.

7. SEISMIC ANALYSIS & STRENGTHENING

Analysis has indicated that the existing concrete shear walls are overstressed to varying degrees in shear and overturning in both directions of the building. It is our recommendation that the building be seismically strengthened in both directions by adding new concrete shear walls and foundations at the exterior walls on the four sides of the building as shown on Figure 30. This approach is similar to option 1A of the BPLW seismic study, except walls are added in both directions rather than just the east-west direction. The location of the shear walls shown on Figure 30 is conceptual only and the shear walls could be moved along the grids on which they are located if a different wall location is more desirable. A conceptual elevation of the proposed new shear walls is shown on Figure 31.

The economy of this method of strengthening coupled with the effectiveness and stiffness compatibility of the new shear walls are the main reasons that we recommend this option. The new shear walls will need to be connected to the existing floor and roof slabs as shown schematically on Figure 32. Removal of the existing unreinforced masonry walls will be required where the new concrete shear walls are placed to complete this connection and place the new shear walls. The

existing radiator units at these walls will need to be removed and relocated as part of this construction. The thickness of the new concrete shear walls varies from 12" thick at the roof level to 20" thick at the basement level as shown on the shear wall elevation. The new shear walls were designed using these thicknesses to limit the shear and overturning stresses in the existing shear walls to allowable levels. Preliminary design of the new shear walls is based on a concrete strength f'_c of 5,000 psi. Some limited additional bending reinforcing may be required at the existing shear walls depending on the results of final design analysis, but this does not appear to be necessary based on the results of our preliminary analysis.

Another advantage of the shear wall method of strengthening is that the stiffness of the new shear walls limits lateral drift of the building during an earthquake. Decreased lateral drift limits the bending forces imposed on the building columns and floor slabs during the earthquake. Because bending forces are limited, the associated additional shear forces imposed by the bending forces will be limited. Because the punching shear capacity of the floor slabs is already insufficient, any additional shear forces could cause failure of the floor slabs at the columns. Because the shear wall forces are limited under the shear wall method of strengthening, the possibility of this type of shear failure is reduced.

New footings will be required below each of the new shear walls to support the weight of the new shear walls and more importantly, resist the overturning forces imposed on the new shear walls during an earthquake. The size and design parameters of the new footings are dependent on the findings of a geotechnical report which is beyond the scope of this study. A schematic detail of the new footings is shown on Figure 33. The existing unreinforced masonry exterior walls should be anchored to the structure as shown on Figure 34 to reduce the possibility of these walls falling from the building during an earthquake. The estimated cost to complete the seismic upgrade of the building is shown in section 10. The estimated costs shown in section 10 assume that the seismic upgrade will be completed at the same time as floor slab reinforcing and floor topping work is being performed. Costs for demolition and reconstruction of finishes are included only when they are unique to the seismic upgrade and would not need to be performed under the floor slab reinforcing or floor topping work. If the seismic upgrade is completed at a different time, the cost of the upgrade will increase due to the added cost for demolition and reconstruction of finishes.

By constructing the new concrete shear walls and foundations at the exterior of the building, the disruption to the tenants required by the seismic strengthening would be fairly minor. Tenants within approximately 8 to 10 feet of the areas where new shear walls would be constructed would need to be relocated, but the remainder of the tenants could remain in place. If tenants remain in the building, a significant amount of after hours work would most likely be required because of construction noise. Extensive drilling and chipping of existing concrete would be required to connect the new shear walls to the existing floor and roof slabs. If the seismic upgrade is constructed at the same time as other floor strengthening measures, the disruption to the tenants could be reduced from that which would be required if the seismic upgrade and floor strengthening were constructed at different times. There would also be some economy of scale which would result in lower total costs if the seismic upgrade and strengthening measures were constructed at the same time. Disruption caused by the anchorage of the existing exterior walls to the structure should be fairly minor if the work is performed after hours. Some limited relocation of tenants will be required to place the angle at the interior of the exterior walls, but if this work is performed in sections, the period of the relocation should be fairly short.

8. STRENGTHENING MEASURES

Where punching shear capacity limits the live load capacities of the floor slabs in the upper levels of the building, the live load capacity can be increased by adding new column capitals at the top of the existing columns directly below the concrete floor slab as shown in Figure 24. Another option would be to completely jacket the existing concrete columns with new reinforced concrete to increase the column size as required to provide sufficient punching shear and live load capacity. Since the flexural capacity of the floors appears to be sufficient to support the original 80 psf design live load, it is recommended that any new punching shear reinforcing be constructed so the full design live load can be used rather than reinforcing the floors to some lower level such as the code minimum of 50 psf. The estimated cost for upgrading the floor live load capacity to the original design live load by adding new concrete column capitals is shown in section 10. The placing of concrete in the column capitals presents some challenges associated with getting concrete into the building. Existing finishes will need to be protected where pump hoses run through

the building. Tenants will need to be temporarily relocated where work is taking place. Temporary relocation of tenants may also be required for safety reasons in areas where pump hoses are run through the building.

Another option that could be used to increase the punching shear and live load capacity of the floor slabs would be to construct new structural steel collars around the existing concrete columns below the floor slabs as shown on Figure 25. Some shimming or epoxy grouting between the new steel collar and the existing slab will probably be required to provide full contact between the slab and collar.. New structural steel columns at the four sides of the existing concrete columns could be used to support the new steel collar. The steel collar could be loaded against the bottom surface of the floor slab by lifting the steel columns using threaded studs and nuts below the column base plates as shown on Figure 25. Once lifting operations are complete, the steel columns can be anchored to the existing concrete columns to transfer loads into the concrete columns. This option has the advantage of not having to place concrete inside the building and will eliminate some of the problems associated with the concrete placement. It would be easier and cleaner to bring the structural steel into the building. Field welding would be required to complete the steel collars below the slab and drilling of the existing concrete would produce dust. Dust walls would need to be constructed around the columns where work was being performed. The estimated cost to upgrade the floor live load capacity to the original design live load by adding new steel column collars is shown in section 10.

Portions of the existing ceilings will need to be demolished and replaced in order to place the new column capitals or column collars. Portions of the existing partition walls will also need to be demolished and replaced at areas adjacent to the columns. Limited removal and replacement of existing services such as mechanical ducts, electrical lines, lights, etc., will be required in the areas adjacent to the columns. Construction of the new concrete column capitals or steel column collars along with the associated finish demolition and replacement will require that tenants be relocated during the work. The relocation effort required to strengthen the existing floor slabs to support code minimum live loads will be less than that required to strengthen the floor slabs to support original design live loads due to the lower number of columns that would require column capitals or collars. Costs to relocate tenants, furniture, etc. have not been included in the cost estimate of section 10. Hammer drilling of holes to place new anchors or rebar dowels, and the roughening of existing concrete

will need to be done after hours to keep from disturbing tenants on other floors of the building. If all tenants are removed from the building, such after hours work will not be required.

Analysis has indicated that the deflections in the existing concrete floor slabs are consistent with calculated long term deflections, and as such, are a serviceability problem and not a strength problem. Flexural reinforcing appears to be sufficient to support the design live load of 80 psf. The live load limitations are due to reduced punching shear capacity rather than reduced flexural capacity of the slabs. Remedial measures at the concrete slabs are optional and are not required to increase the capacity of the floor slabs. Any remedial measures will provide a more serviceable building and a perception of higher quality in the building. Even if the punching shear capacity of the slabs is increased and other remedial measures are completed, the loading on the floors should continue to be carefully monitored and controlled. There are enough questions about the floor slabs in the building to mandate conservatism.

Two methods to reduce deflections in the concrete floor slabs have been considered. The first method would use new cementitious topping on the existing concrete slabs to provide a more level floor surface. The existing carpet, partition walls, ceilings, and vinyl-asbestos tile would need to be removed to expose the existing concrete floor slabs to place the new topping. A bonding compound could be used to bond new topping to the existing slabs. It has been assumed that the existing partition walls would be demolished so the new topping could be placed in a continuous pour without interruptions. If the new topping were placed without demolishing the existing partition walls, a void or trench would exist in the topping when the partition walls were demolished during future remodeling. The existing partition walls do not appear to be seismically braced as discussed in section 6. The new partition walls would be properly braced and reduce the non-structural damage during an earthquake. Demolition of the existing partition walls will require that major portions of the existing ceilings be demolished and reconstructed. Because the existing ceilings require seismic bracing as discussed in section 6, it is recommended that all ceilings be demolished and replaced with a new acoustical ceiling tile that is properly braced.

The existing junction box covers and connection boxes for the in-floor electrical duct system would need to be raised so they are flush with the new floor surface. Tenants would need to be relocated from areas where work associated with demolition of existing finishes, placement of new topping on the existing floor slabs, and reconstruction of finishes is underway. If tenants remain in other portions of the building during the work, some after hours work will probably be required depending on the requirements of the building manager. Hammering and drilling operations should be fairly minor under this option, but there will be some noise associated with the demolition operations and placement of the new topping.

Once the new topping system was complete, new partition walls and ceilings could be constructed, and new floor coverings could be placed over the topping to complete the finish of the floor. The cost to complete the new topping on the existing slabs and reinforce the floor slabs by adding new concrete column capitals is shown in section 10. A disadvantage of this topping system is that it reduces the live load capacity of the floor slabs proportional to the amount of topping required to level the floor. If new column capitals or column collars were constructed to bring the live load capacity of the existing floor slabs up to the design level of 80 psf, the addition of the new topping would reduce the average live load capacity of the floor slabs to approximately 60 to 70 psf based on the average slab deflections measured at the center of the bays and the reinforcing provided in the slabs. Long term deflections should be complete by now, so the only change in deflection should be due to the change in the magnitude of live load on the floor slabs caused by the addition of the leveling material. This elastic deflection should be quite small.

If new topping is placed on the existing floor slabs, it is recommended that the live load capacity of the floor slabs be increased by adding new concrete column capitals below the floor slabs. The problems associated with bringing concrete into the building will not be a consideration due to the extensive demolition and reconstruction work at each floor. A significant amount of cementitious topping material will need to be placed at each floor, and placement of concrete at the column capitals will not cause additional problems. The estimated cost to place the new concrete column capitals at the same time the topping is being placed would be less than placing new structural steel collars to increase the live load capacity of the floor slabs.

The second method considered to reduce the deflections in the existing concrete floor slabs, is the addition of new steel beams on all interior grid lines between columns to support the floor slabs. New steel beams would not be required at the perimeter of the building where the concrete spandrel beam exists. New steel columns alongside the existing concrete columns could be used to support the new steel beams. The existing concrete slabs could be lifted by jacking the new steel beams and columns using the floor below as a support. Steel shims between the existing concrete slabs and new steel beams could be used to compensate for deflections in the steel beams and provide a fairly level support for the existing concrete slabs when jacking is completed. Once the lifting operations are completed, the new steel columns can be anchored to the existing concrete columns to transfer loads into the existing concrete columns. See Figures 28 and 29 for schematic drawings of this method.

A major disadvantage of this method is the constriction of the space between the bottom of the floor slabs and the ceilings for air ducts, electrical lines, lights and other components. The floor to floor heights are only 11'-0" which leaves a fairly small space between the ceiling and the bottom of the slab. If 16 inches of this space is taken for the new beams on grid lines, approximately 8 inches of space remains between the bottom of the beams and the top of the light fixtures at the ceiling assuming an 8'-0" high ceiling and 4" deep lights. This small space between the new beams and the ceiling will make air distribution especially difficult if not impossible.

Another major disadvantage to this method of floor support is the expense needed to complete construction of steel beams. Preliminary cost estimates indicated a cost of approximately \$12,500,000 to complete this option so it was dropped from consideration and not studied further. Because of the extensive work required under this option, tenants would need to be relocated during the strengthening work. It would be possible to phase the construction to work on one, two, or more floors at a time with tenants remaining in the floors where work is not in progress. If all tenants are removed from the building, after hours work will not be required.

9. PRIORITIZATION OF STRENGTHENING MEASURES

Life Safety Issues

Life safety issues are considered to be those strengthening measures that are essential to the safety of the building occupants and should be completed in the next 6 months to one year. We would recommend that the live load and associated punching shear capacity of the floor slabs be increased by constructing the new concrete column capitals or steel column collars at the existing columns as required to provide the original design live load capacity of 80 psf. Where existing live load capacities are less than the code minimum office live load of 50 psf, we recommend that slab reinforcing be completed as soon as is feasibly possible. The reinforcing of the floor slabs should begin at levels 7 and 8 and work downward to level 4. A very conscious effort should be made in the meantime to limit live loads so they are within the allowable live load magnitudes shown on Figures 12 through 17. The live loads at level 8 have been inventoried and studied at level 8 as part of another study. The live loads have been reduced as required by this study and must not be increased until reinforcing work is complete. Even after all reinforcing work has been completed in the building, live loads should be monitored to keep live loads within allowable magnitudes.

Short To Long Term Issues

Short to long term issues are considered to be those strengthening measures which could be acceptably accomplished within the next five years or more. We would recommend that the seismic upgrade of the building be completed in order to provide an increased level of safety to the building occupants. The timing of the seismic upgrade is dependent on funding and prioritization with respect to other buildings in the GSA inventory. Since the building is located in seismic zone 2, it may be advisable to complete a seismic upgrade on a building in seismic zone 4 or 3 before seismically upgrading this building. The timing of any seismic upgrade will need to be decided by the owner based on risk analysis and funding constraints.

Optional Issues

Optional issues are those remedial measures which are not required, or would not provide an increased level of safety or load capacity in the building. These remedial measures are concerned with serviceability and perception issues in the building. We would recommend that the measures to decrease the deflections in the existing floor

slabs be included under optional remedial measures. These measures would provide a perception of higher quality in the floors of the building, and would reduce some of the problems in the building associated with the present floor deflections. Optional remedial measures could be considered to be a short term issue, a long term issue, or no issue at all depending on financial considerations and the perceptions of occupants in the building.

10. COST ESTIMATES

The costs that follow reflect not only the structural costs, but also the demolition, repair and/or replacement of finishes or architectural elements which are directly related to the strengthening measures. Allowances have also been included to cover the cost for demolition, realignment and/or replacement of mechanical equipment and ducts, plumbing, electrical wiring and equipment, etc. The costs are a preliminary estimate only, and may increase due to contingency items, changes in the construction market and other unforeseen circumstances. The costs could also change as further study regarding constraints on constructions, phasing, and occupant relocation is completed. The costs for the various strengthening and remedial measures are summarized below according to CSI divisions.

INCREASE FLOOR LIVE LOAD CAPACITY WITH NEW CONCRETE CAPITALS

<u>CSI DIVISION</u>	<u>AMOUNT</u>
DIVISION ONE - GENERAL CONDITIONS	\$483,945
DIVISION TWO - SITEWORK	\$184,900
DIVISION THREE - CONCRETE	\$146,650
DIVISION NINE - FINISHES	\$95,620
DIVISION FIFTEEN - MECHANICAL	\$32,700
DIVISION SIXTEEN - ELECTRICAL	<u>\$21,800</u>
TOTAL	\$965,615

INCREASE FLOOR LIVE LOAD CAPACITY WITH NEW STEEL COLLARS

<u>CSI DIVISION</u>	<u>AMOUNT</u>
DIVISION ONE - GENERAL CONDITIONS	\$510,387
DIVISION TWO - SITEWORK	\$94,900
DIVISION FIVE - STEEL	\$292,730
DIVISION NINE - FINISHES	\$95,620
DIVISION FIFTEEN - MECHANICAL	\$32,700
DIVISION SIXTEEN - ELECTRICAL	<u>\$21,800</u>
TOTAL	\$1,048,137

PLACE TOPPING ON EXISTING FLOOR TO REDUCE DEFLECTIONS AND
PLACE NEW CONCRETE COLUMN CAPITALS

<u>CSI DIVISION</u>	<u>AMOUNT</u>
DIVISION ONE - GENERAL CONDITIONS	\$3,389,515
DIVISION TWO - SITEWORK	\$1,225,000
DIVISION THREE - CONCRETE	\$1,411,650
DIVISION NINE - FINISHES	\$2,126,500
DIVISION FIFTEEN - MECHANICAL	\$396,000
DIVISION SIXTEEN - ELECTRICAL	<u>\$440,000</u>
TOTAL	\$8,988,665

SEISMIC STRENGTHENING

<u>CSI DIVISION</u>	<u>AMOUNT</u>
DIVISION ONE - GENERAL CONDITIONS	\$1,329,626
DIVISION TWO - SITEWORK	\$244,121
DIVISION THREE - CONCRETE	\$858,700
DIVISION FIVE - STEEL	\$240,000
DIVISION SEVEN - THERMAL PROTECTION	\$14,942
DIVISION EIGHT - DOORS & WINDOWS	\$55,008
DIVISION NINE - FINISHES	\$90,691
DIVISION TWELVE - FURNISHINGS	\$4,767
DIVISION FIFTEEN - MECHANICAL	\$48,000

DIVISION SIXTEEN - ELECTRICAL

\$6,400

TOTAL

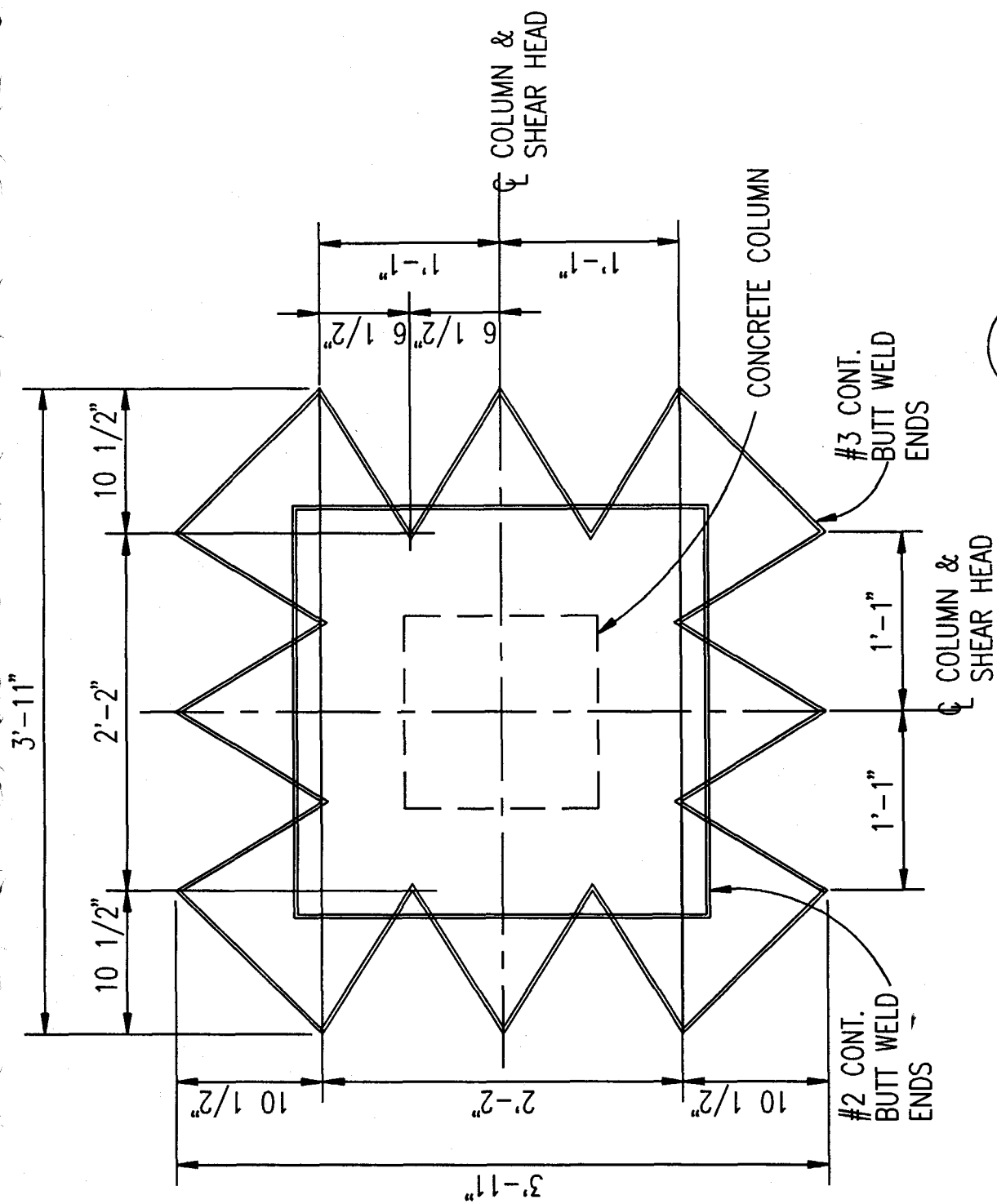
\$2,892,255

11. COST BASIS

The costs for the described strengthening and remedial measures addressed in this report have been based on certain assumptions. Because of the potential high cost of the strengthening and remedial measures, it is important to understand the assumptions which were made in arriving at these costs. The occupancy and function of the building could probably be maintained during certain strengthening measures, but other measures would require significant changes in occupancy if not complete vacating of the building. Therefore, there would not only be structural strengthening costs, but there would also be added construction costs due to phasing, the disruption of function, operation, and services. These costs have not been included in the cost estimate and must be added as required. Costs for the demolition and reconstruction of finishes associated with the retrofit construction have been included in the estimated costs.

12. LIMITATIONS

It must be cautioned that the recommendations presented in this report are limited by the extent and accuracy of information available to us during the course of this investigation. Conditions detrimental to structural and non-structural parts of the building may exist which are not documented in the original drawings and were not visible or were not otherwise discovered during the field observation portion of this investigation. These conditions must be considered during the contract documents preparation phase or construction phase as they are discovered. This report is intended for planning purposes and not for construction.



SHEAR HEAD DETAIL IN PLAN

NO SCALE

FIGURE 1

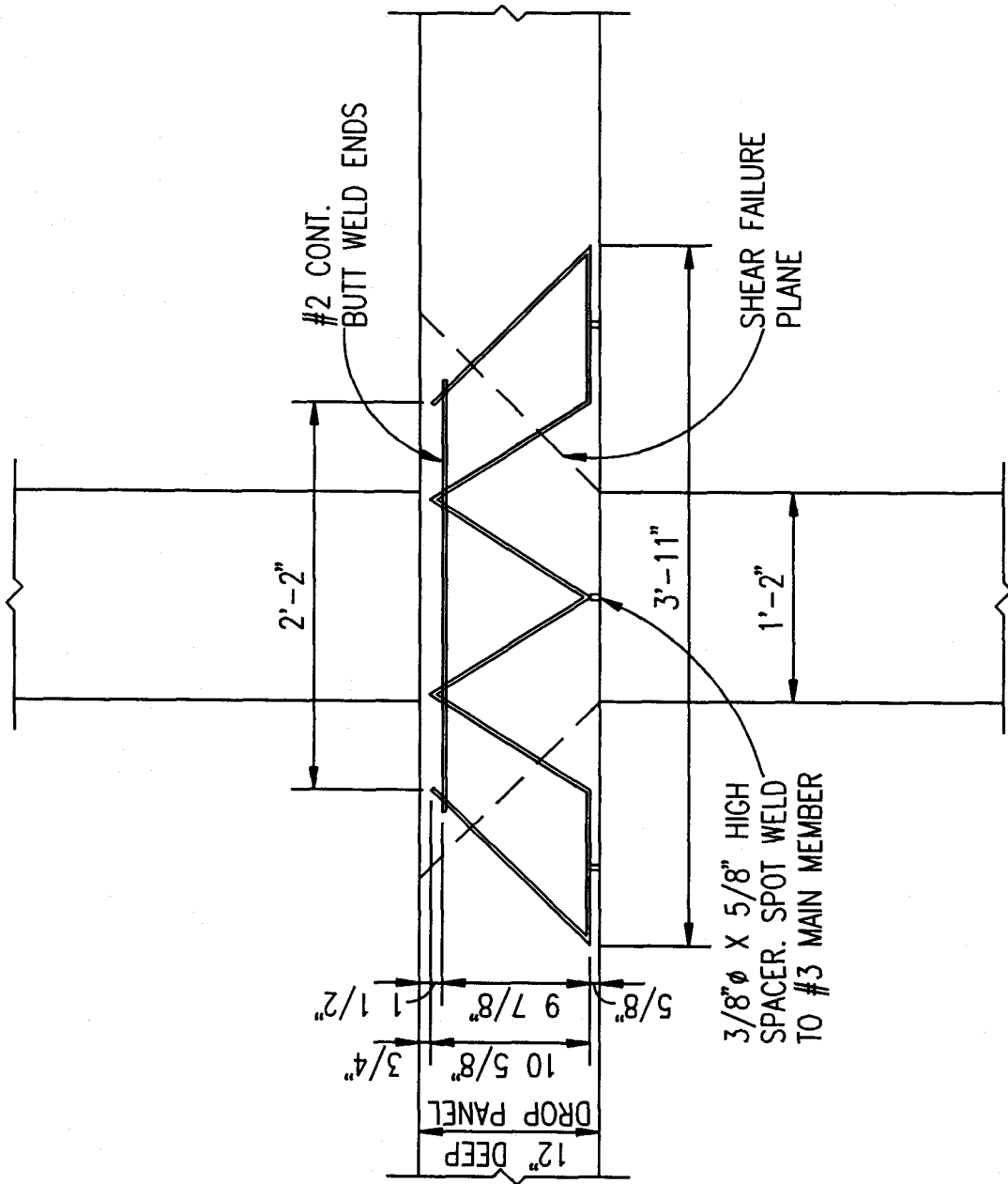
#3 CONT.
BUTT WELD
ENDS

CONCRETE COLUMN

FIGURE 1

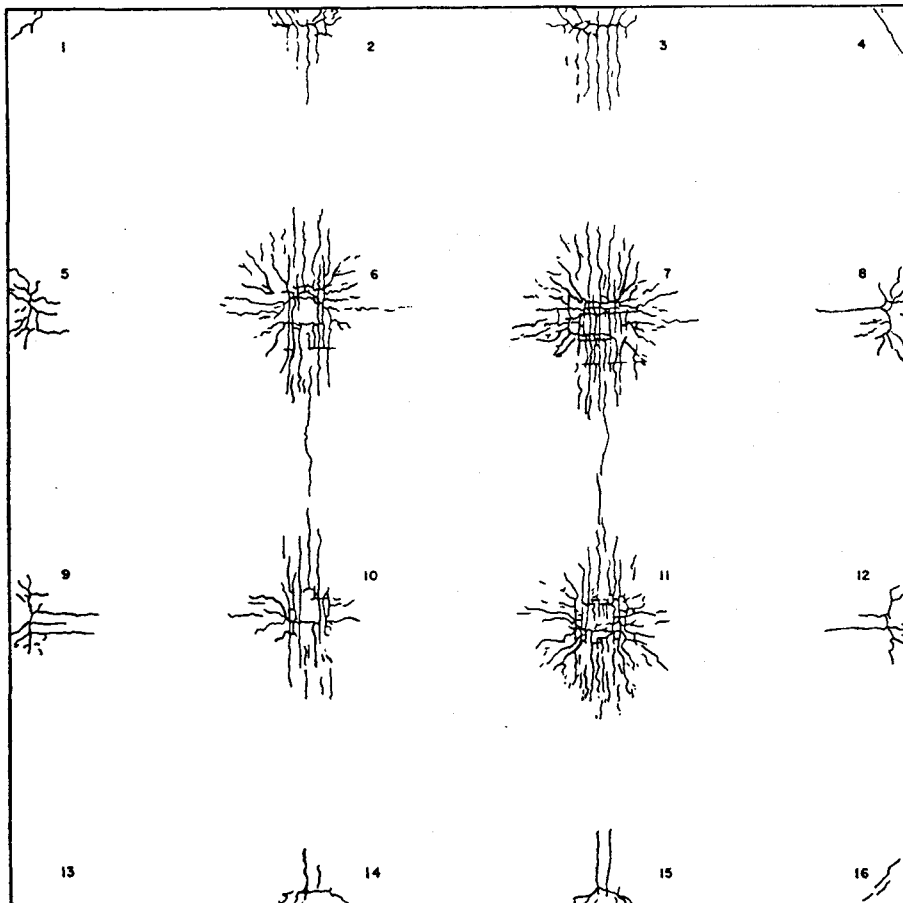
ALBUQUERQUE FEDERAL BUILDING

ALBUQUERQUE FEDERAL BUILDING



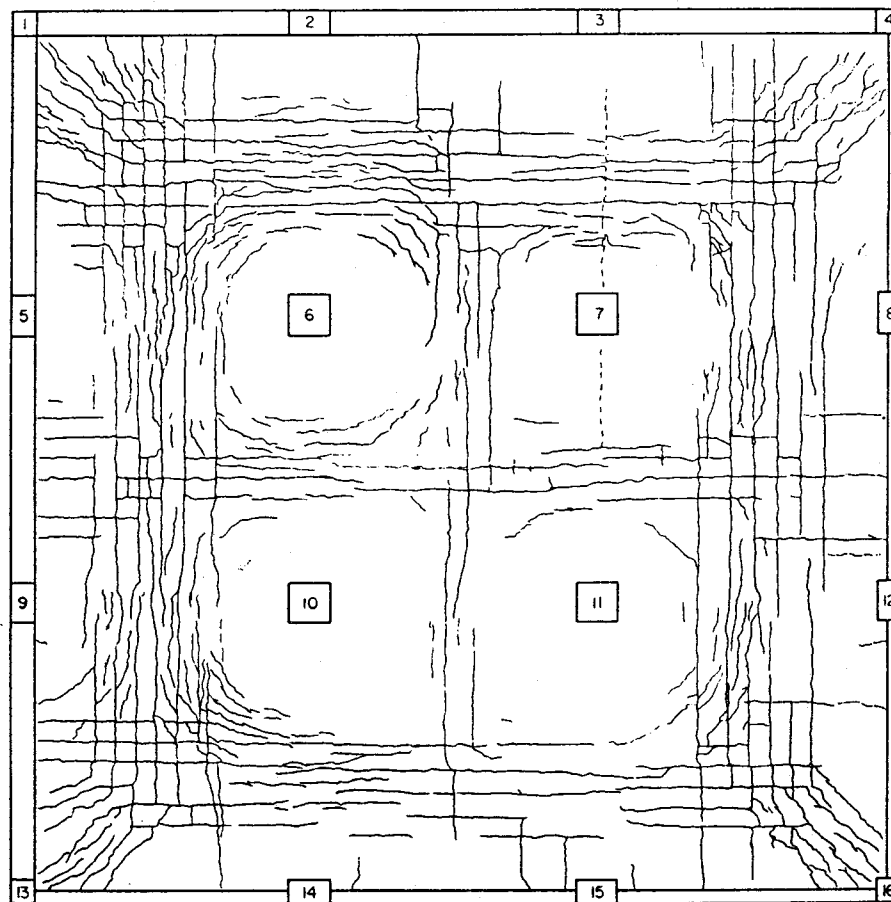
SHEAR HEAD DETAIL IN SECTION
NO SCALE

FIGURE 2
ALBUQUERQUE FEDERAL BUILDING



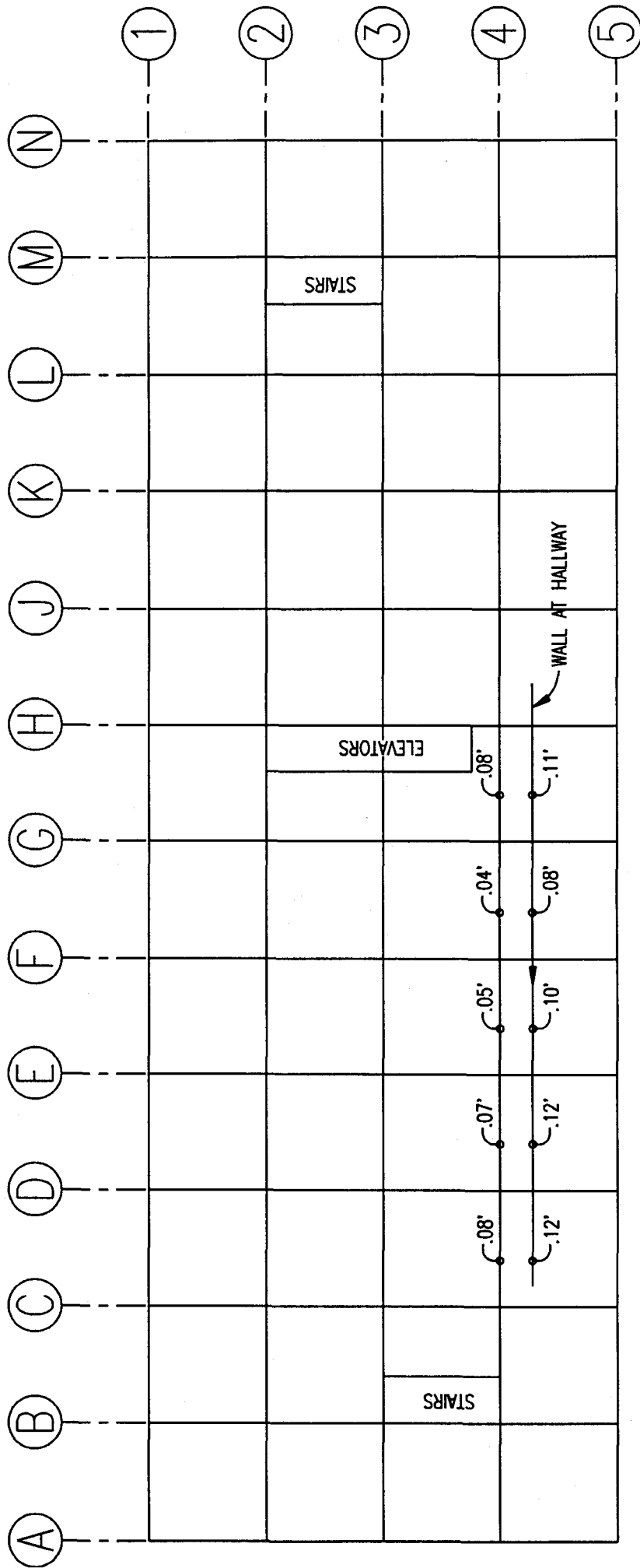
TOP SURFACE CRACK PATTERN AT DESIGN LOAD

FIGURE 3



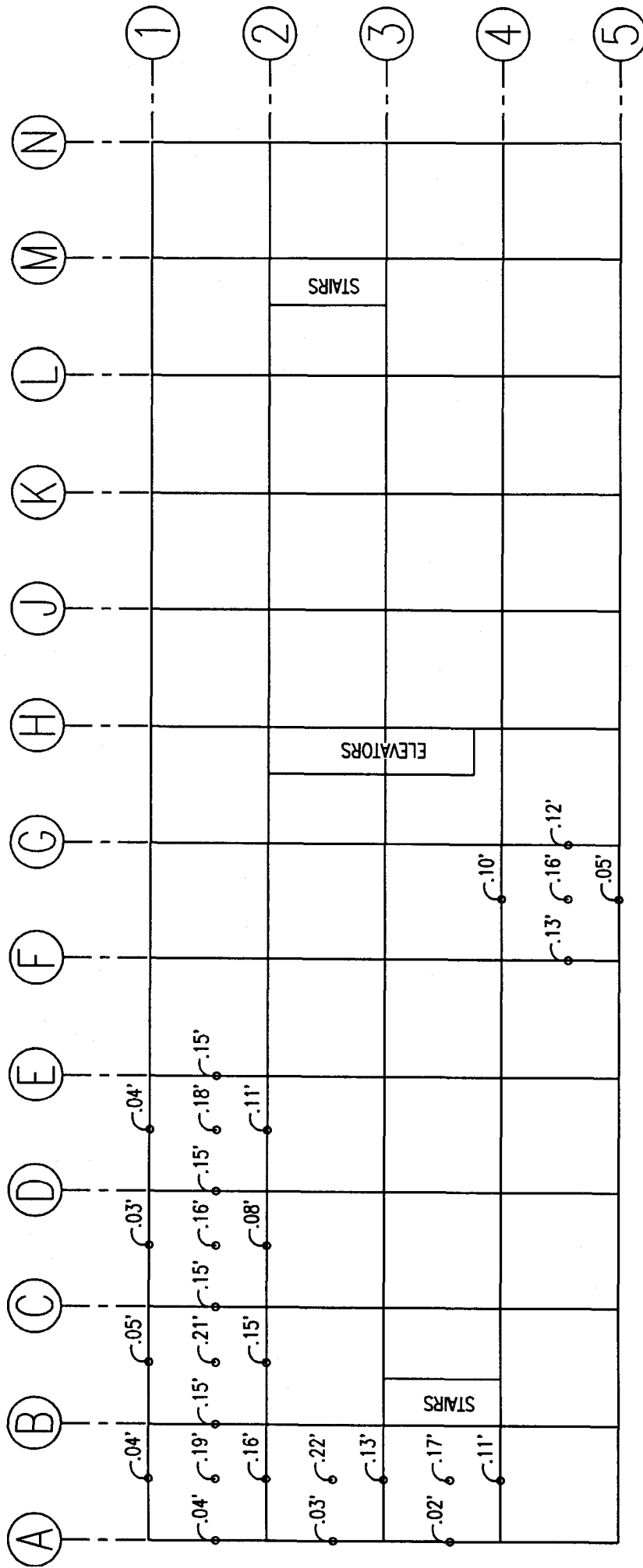
BOTTOM SURFACE CRACK PATTERN

FIGURE 4



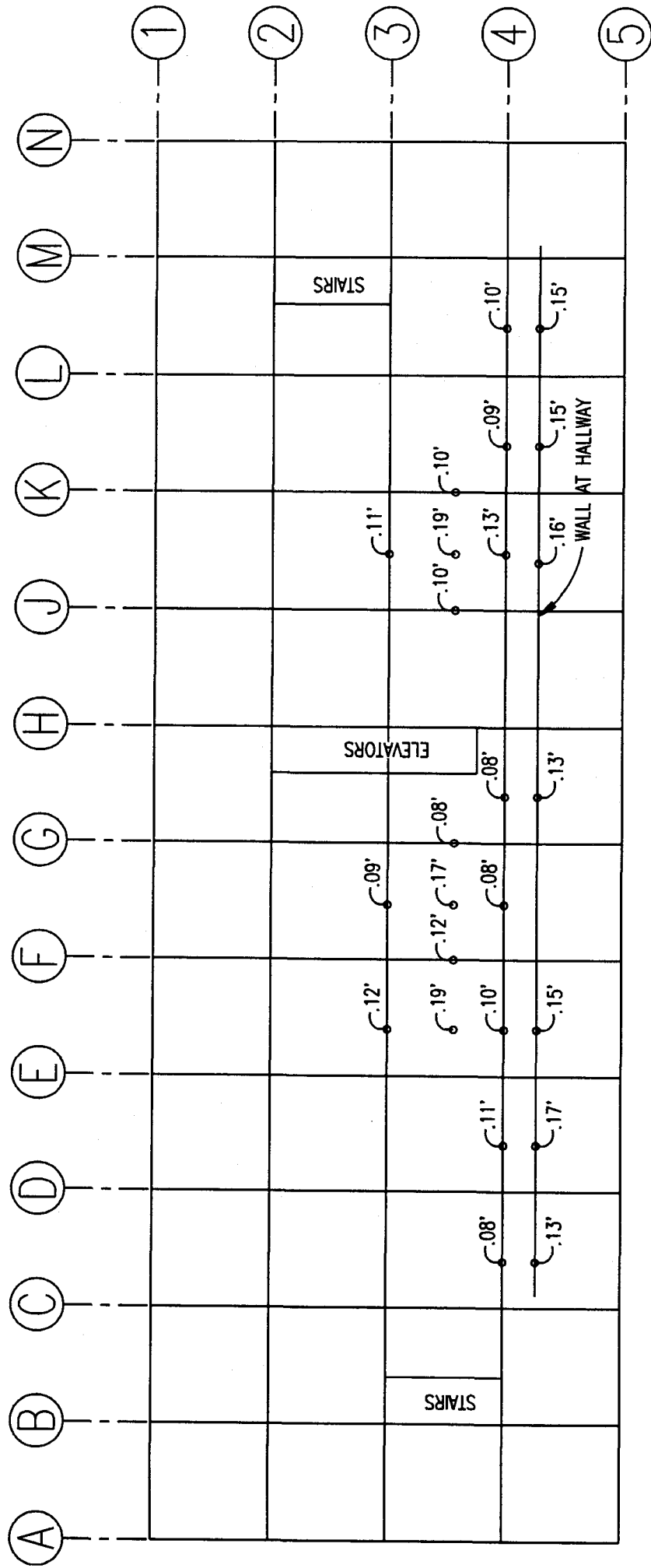
ALBUQUERQUE FEDERAL BUILDING
 FLOOR DEFLECTIONS
 LEVEL 1

FIGURE 5



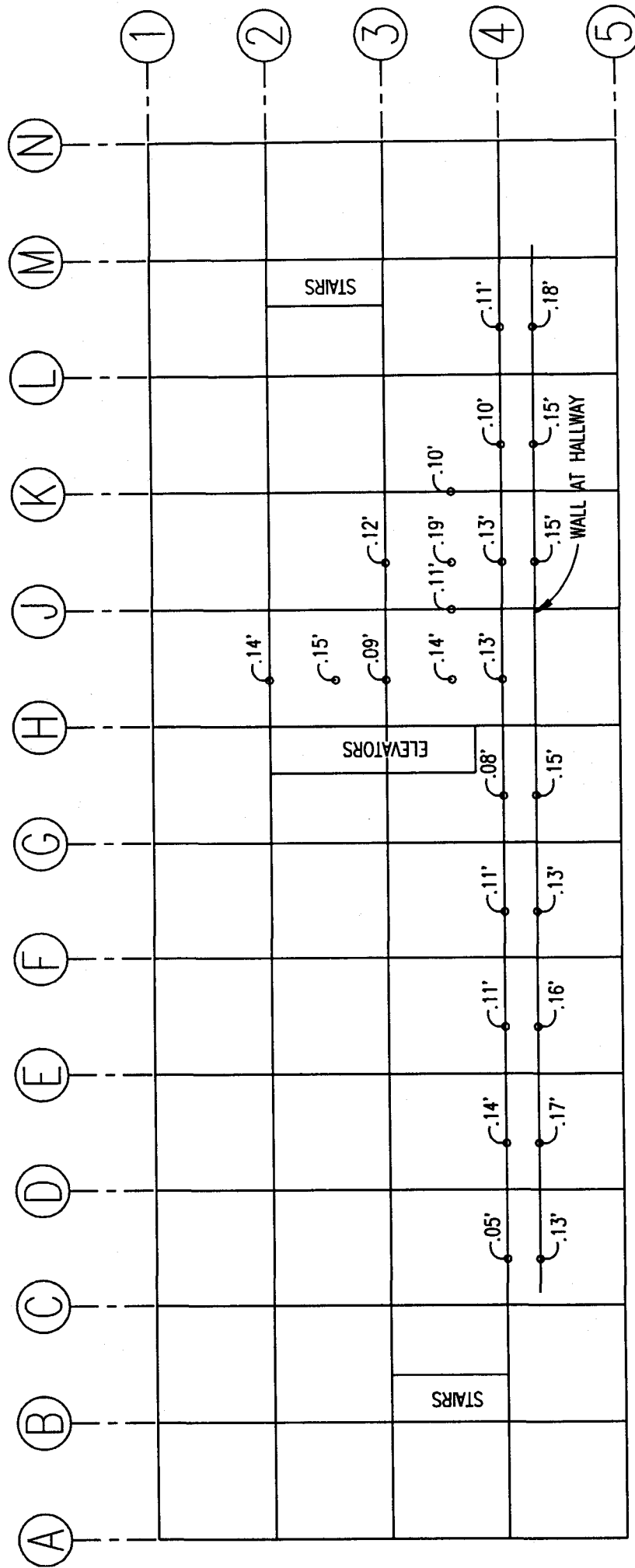
ALBUQUERQUE FEDERAL BUILDING
FLOOR DEFLECTIONS
LEVEL 3

FIGURE 6



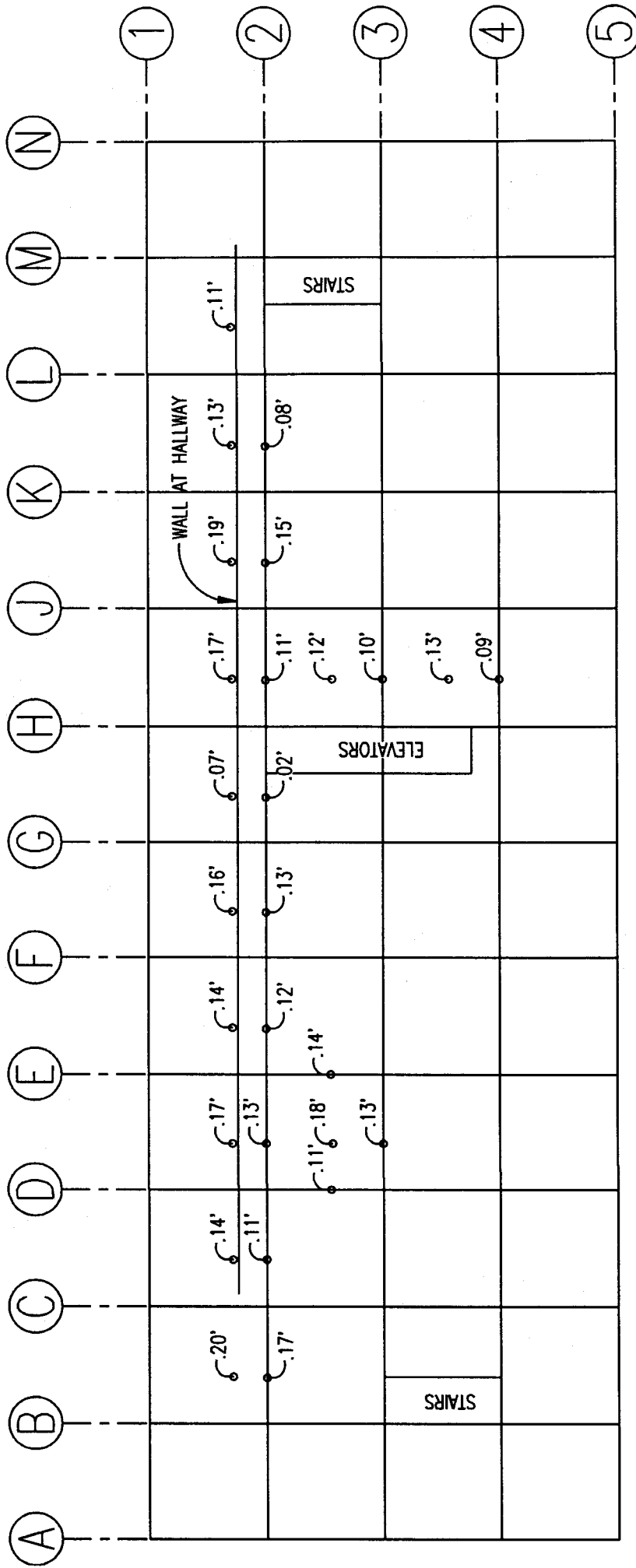
ALBUQUERQUE FEDERAL BUILDING
FLOOR DEFLECTIONS
LEVEL 4

FIGURE 7



ALBUQUERQUE FEDERAL BUILDING
FLOOR DEFLECTIONS
LEVEL 5

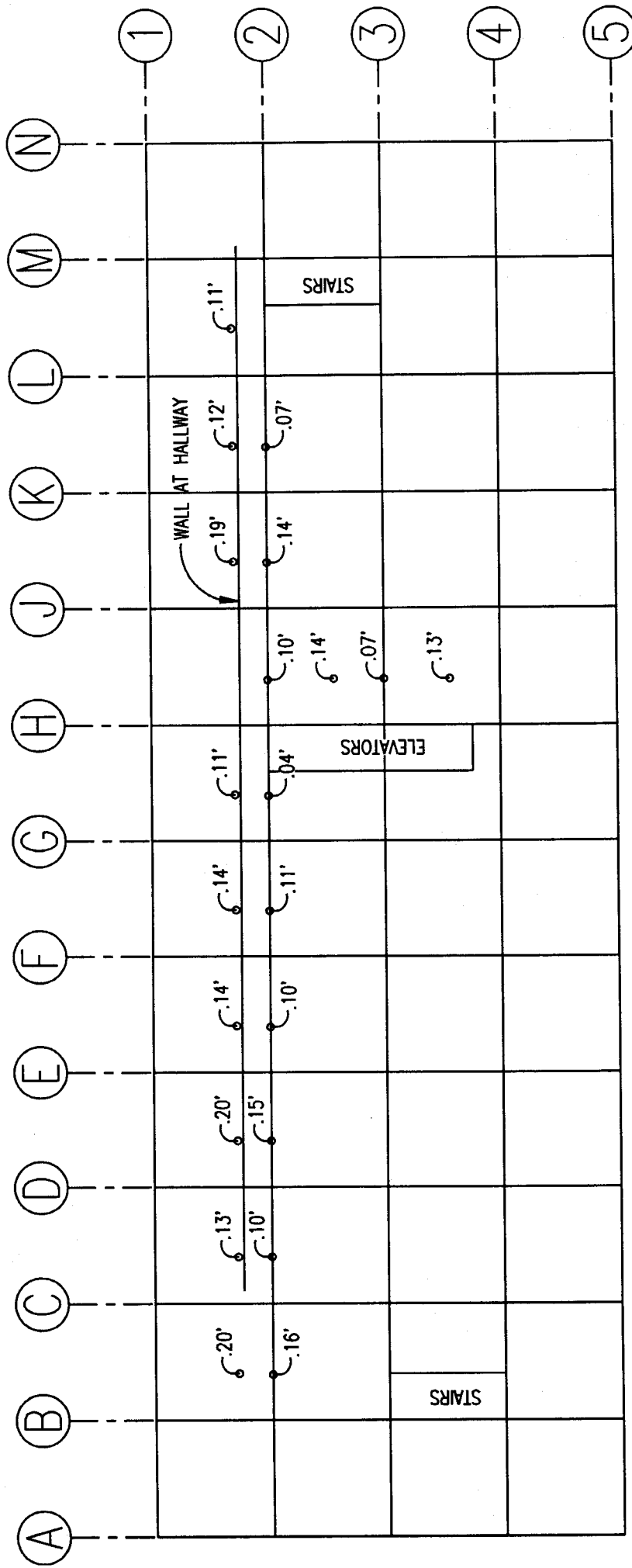
FIGURE 8



ALBUQUERQUE FEDERAL BUILDING
FLOOR DEFLECTIONS

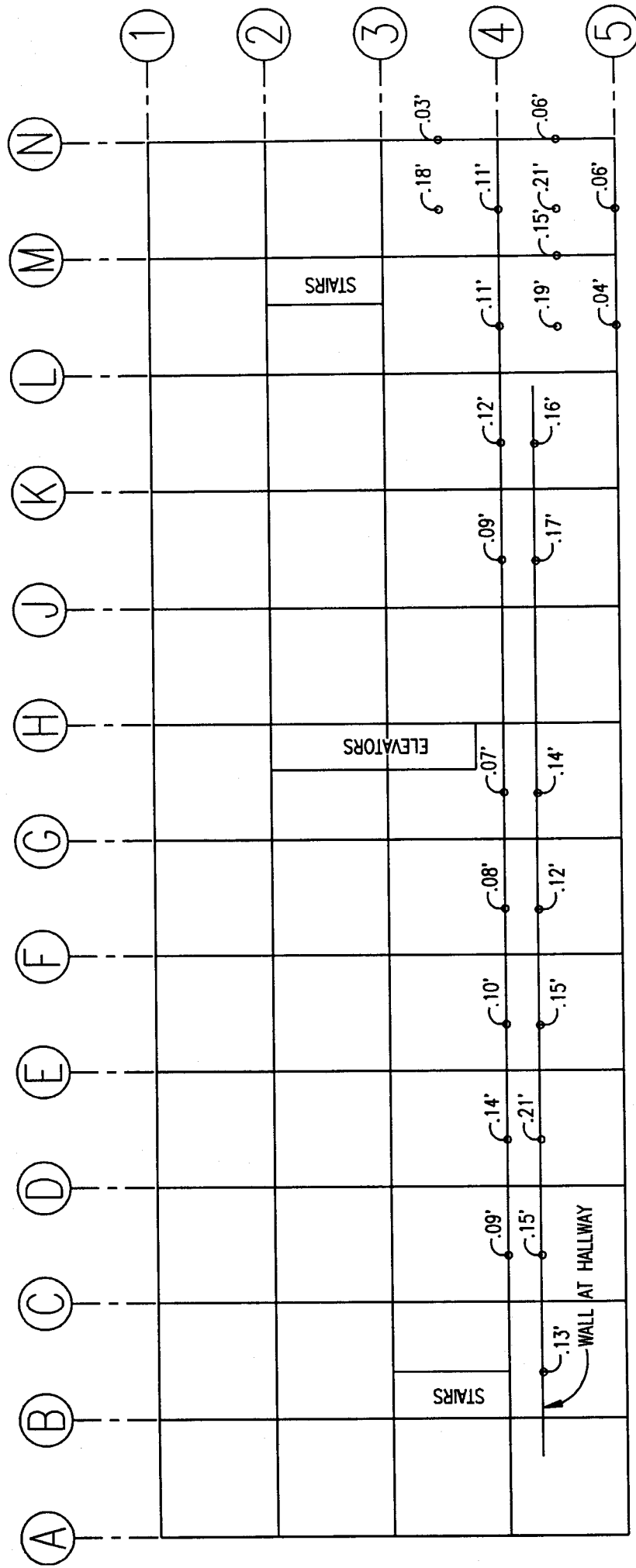
LEVEL 6

FIGURE 9



ALBUQUERQUE FEDERAL BUILDING
FLOOR DEFLECTIONS
LEVEL 7

FIGURE 10



ALBUQUERQUE FEDERAL BUILDING
FLOOR DEFLECTIONS
LEVEL 8

FIGURE 11

(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(J)	(K)	(L)	(M)	(N)	
80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	①
80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	150	STAIRS	80+20	②
80+20	80+20	150	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	③
80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	④
80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	⑤

NOTES:

1. ALLOWABLE LIVE LOADS ARE BASED ON $f'_c = 2,100$ PSI FOR CONCRETE BASED ON TESTING RESULTS.
2. ALL LIVE LOADS IN PSF.
3. 20 PSF FOR MOVABLE PARTITIONS PER UBC REQUIREMENTS.
4. THE FIRST NUMBER AT EACH BAY INDICATES THE ALLOWABLE LIVE LOAD. THE SECOND NUMBER IS THE PARTITION ALLOWANCE.



ALBUQUERQUE FEDERAL BUILDING
ALLOWABLE LIVE LOAD

LEVELS 1-3

FIGURE 12

(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(J)	(K)	(L)	(M)	(N)	
70+20	65+20	65+20	65+20	65+20	65+20	65+20	65+20	65+20	65+20	65+20	80+20	80+20	①
80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	150	STAIRS	80+20	②
80+20	STAIRS	150	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	80+20	③
80+20	80+20	65+20	65+20	65+20	65+20	65+20	65+20	65+20	65+20	65+20	65+20	65+20	④
													⑤

NOTES:

1. ALLOWABLE LIVE LOADS ARE BASED ON $f'_c = 2,100$ PSI FOR CONCRETE BASED ON TESTING RESULTS.
2. ALL LIVE LOADS IN PSF.
3. 20 PSF FOR MOVABLE PARTITIONS PER UBC REQUIREMENTS.
4. THE FIRST NUMBER AT EACH BAY INDICATES THE ALLOWABLE LIVE LOAD. THE SECOND NUMBER IS THE PARTITION ALLOWANCE.



ALBUQUERQUE FEDERAL BUILDING
ALLOWABLE LIVE LOAD

LEVELS 4

FIGURE 13

(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(J)	(K)	(L)	(M)	(N)	
50+20	50+20	50+20	50+20	50+20	50+20	50+20	50+20	60+20	50+20	50+20	80+20	80+20	1
70+20	75+20	75+20	75+20	75+20	75+20	75+20	80+20	75+20	75+20	150	STAIRS	80+20	2
80+20	150	75+20	75+20	75+20	75+20	80+20	80+20	75+20	75+20	75+20	70+20	70+20	3
80+20	80+20	50+20	50+20	50+20	50+20	50+20	50+20	50+20	50+20	50+20	50+20	50+20	4
													5

NOTES:

1. ALLOWABLE LIVE LOADS ARE BASED ON $f'_c = 2,100$ PSI FOR CONCRETE BASED ON TESTING RESULTS.
2. ALL LIVE LOADS IN PSF.
3. 20 PSF FOR MOVABLE PARTITIONS PER UBC REQUIREMENTS.
4. THE FIRST NUMBER AT EACH BAY INDICATES THE ALLOWABLE LIVE LOAD. THE SECOND NUMBER IS THE PARTITION ALLOWANCE.



ALBUQUERQUE FEDERAL BUILDING
ALLOWABLE LIVE LOAD

LEVEL 5

FIGURE 14

A	B	C	D	E	F	G	H	J	K	L	M	N
30+20	40+20	40+20	40+20	40+20	40+20	40+20	40+20	45+20	40+20	40+20	80+20	80+20
50+20	65+20	65+20	65+20	65+20	65+20	65+20	65+20	80+20	65+20	65+20	150	80+20
80+20	STAIRS	150	65+20	65+20	65+20	65+20	65+20	80+20	65+20	55+20	55+20	65+20
80+20	80+20	40+20	40+20	40+20	40+20	40+20	40+20	40+20	40+20	40+20	40+20	40+20

NOTES:

1. ALLOWABLE LIVE LOADS ARE BASED ON $f'_c = 2,100$ PSI FOR CONCRETE BASED ON TESTING RESULTS.
2. ALL LIVE LOADS IN PSF.
3. 20 PSF FOR MOVABLE PARTITIONS PER UBC REQUIREMENTS.
4. THE FIRST NUMBER AT EACH BAY INDICATES THE ALLOWABLE LIVE LOAD. THE SECOND NUMBER IS THE PARTITION ALLOWANCE.



ALBUQUERQUE FEDERAL BUILDING
ALLOWABLE LIVE LOAD

LEVEL 6

FIGURE 15

	(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(J)	(K)	(L)	(M)	(N)	
														(1)
	25+20	25+20	25+20	25+20	25+20	25+20	30+20	35+20	30+20	30+20	30+20	80+20	80+20	(2)
	35+20	35+20	40+20	40+20	40+20	40+20	45+20	55+20	45+20	45+20	150	80+20	80+20	(3)
	80+20	80+20	150	40+20	40+20	40+20	45+20	55+20	45+20	40+20	35+20	40+20	40+20	(4)
	80+20	80+20	25+20	25+20	25+20	25+20	30+20	30+20	30+20	30+20	25+20	30+20	30+20	(5)

NOTES:

1. ALLOWABLE LIVE LOADS ARE BASED ON $f'_c = 2,100$ PSI FOR CONCRETE BASED ON TESTING RESULTS.
2. ALL LIVE LOADS IN PSF.
3. 20 PSF FOR MOVABLE PARTITIONS PER UBC REQUIREMENTS.
4. THE FIRST NUMBER AT EACH BAY INDICATES THE ALLOWABLE LIVE LOAD. THE SECOND NUMBER IS THE PARTITION ALLOWANCE.



ALBUQUERQUE FEDERAL BUILDING
ALLOWABLE LIVE LOAD

LEVEL 7

FIGURE 16

(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(J)	(K)	(L)	(M)	(N)	
10+20	10+20	10+20	10+20	10+20	10+20	10+20	20+20	25+20	25+20	25+20	80+20	80+20	1
20+20	20+20	20+20	20+20	20+20	20+20	20+20	35+20	35+20	35+20	35+20	150	80+20	2
80+20	80+20	20+20	20+20	20+20	20+20	20+20	35+20	35+20	35+20	25+20	25+20	30+20	3
80+20	80+20	10+20	10+20	10+20	10+20	10+20	20+20	20+20	25+20	15+20	15+20	20+20	4
													5

NOTES:

1. ALLOWABLE LIVE LOADS ARE BASED ON $f'_c = 2,100$ PSI FOR CONCRETE BASED ON TESTING RESULTS.
2. ALL LIVE LOADS IN PSF.
3. 20 PSF FOR MOVABLE PARTITIONS PER UBC REQUIREMENTS.
4. THE FIRST NUMBER AT EACH BAY INDICATES THE ALLOWABLE LIVE LOAD. THE SECOND NUMBER IS THE PARTITION ALLOWANCE.



ALBUQUERQUE FEDERAL BUILDING
ALLOWABLE LIVE LOAD

LEVEL 8

FIGURE 17

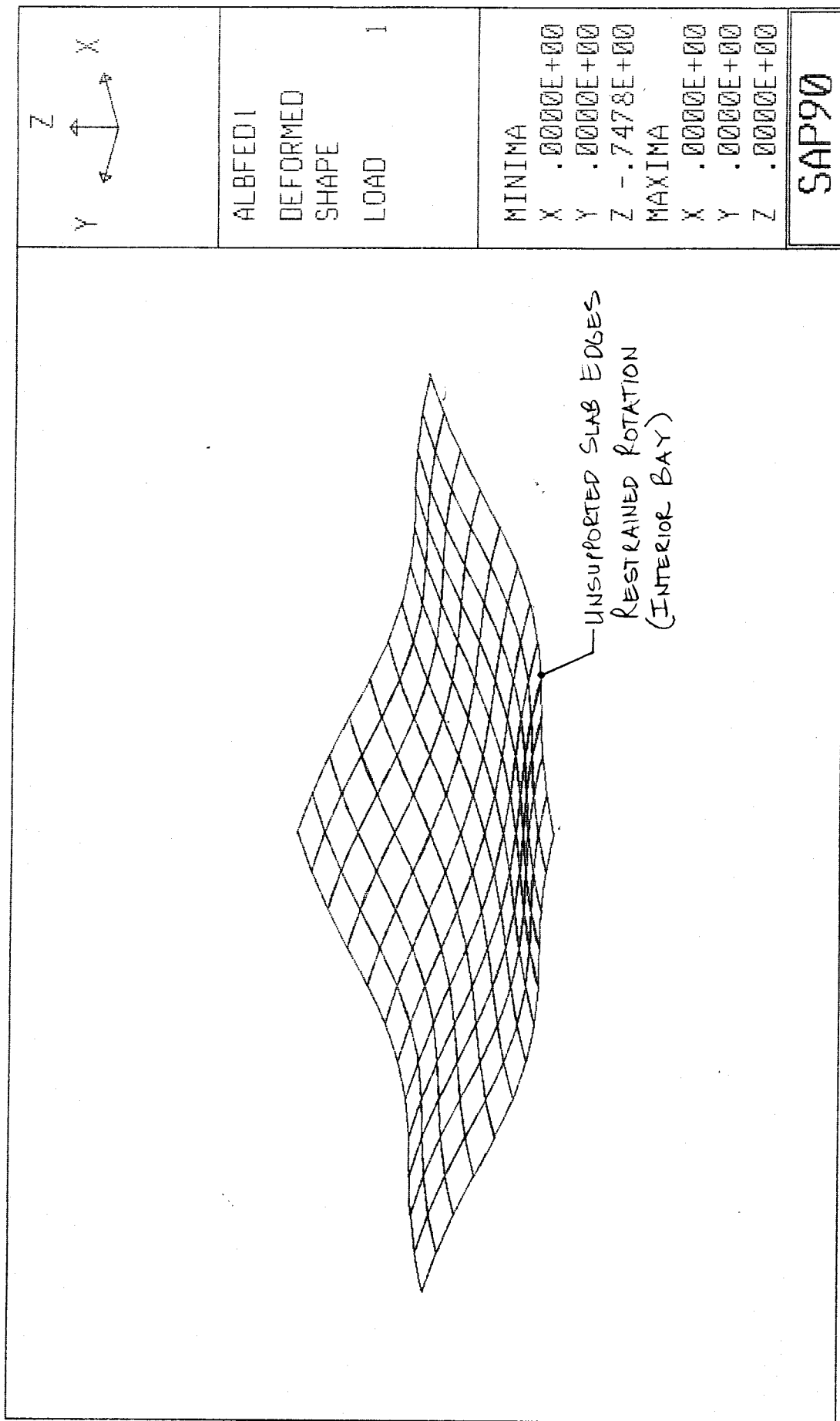


FIGURE 18

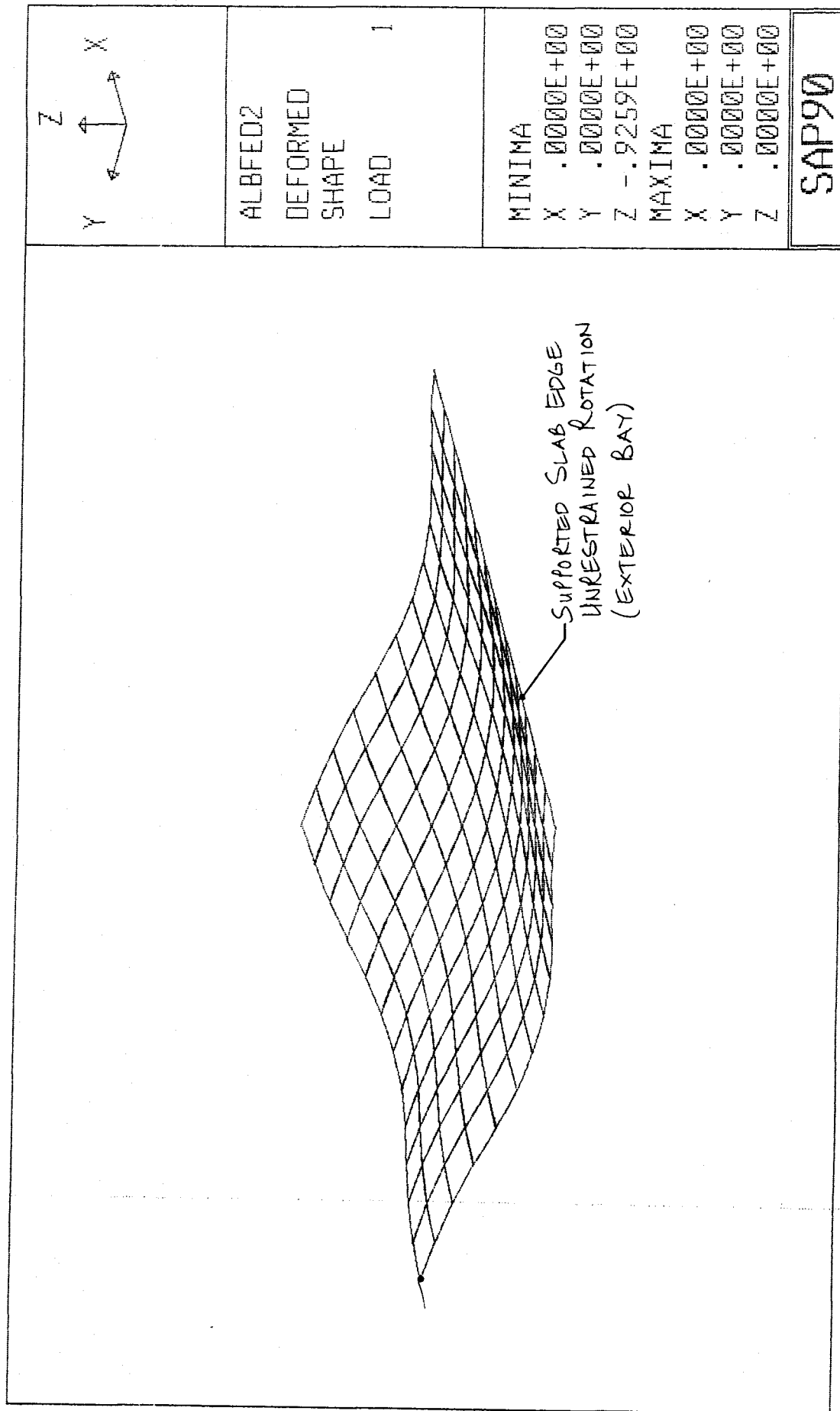


FIGURE 19

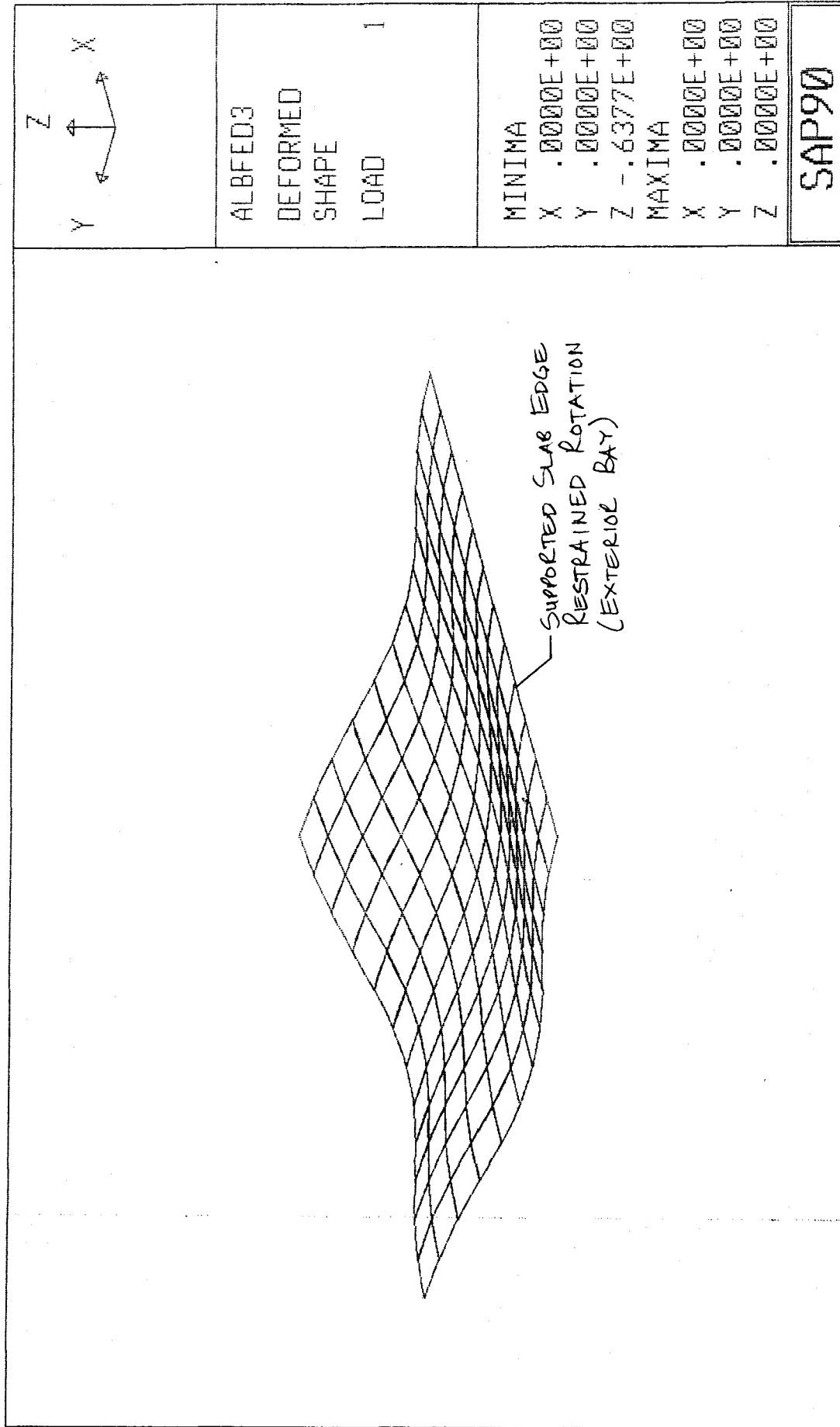


FIGURE 20

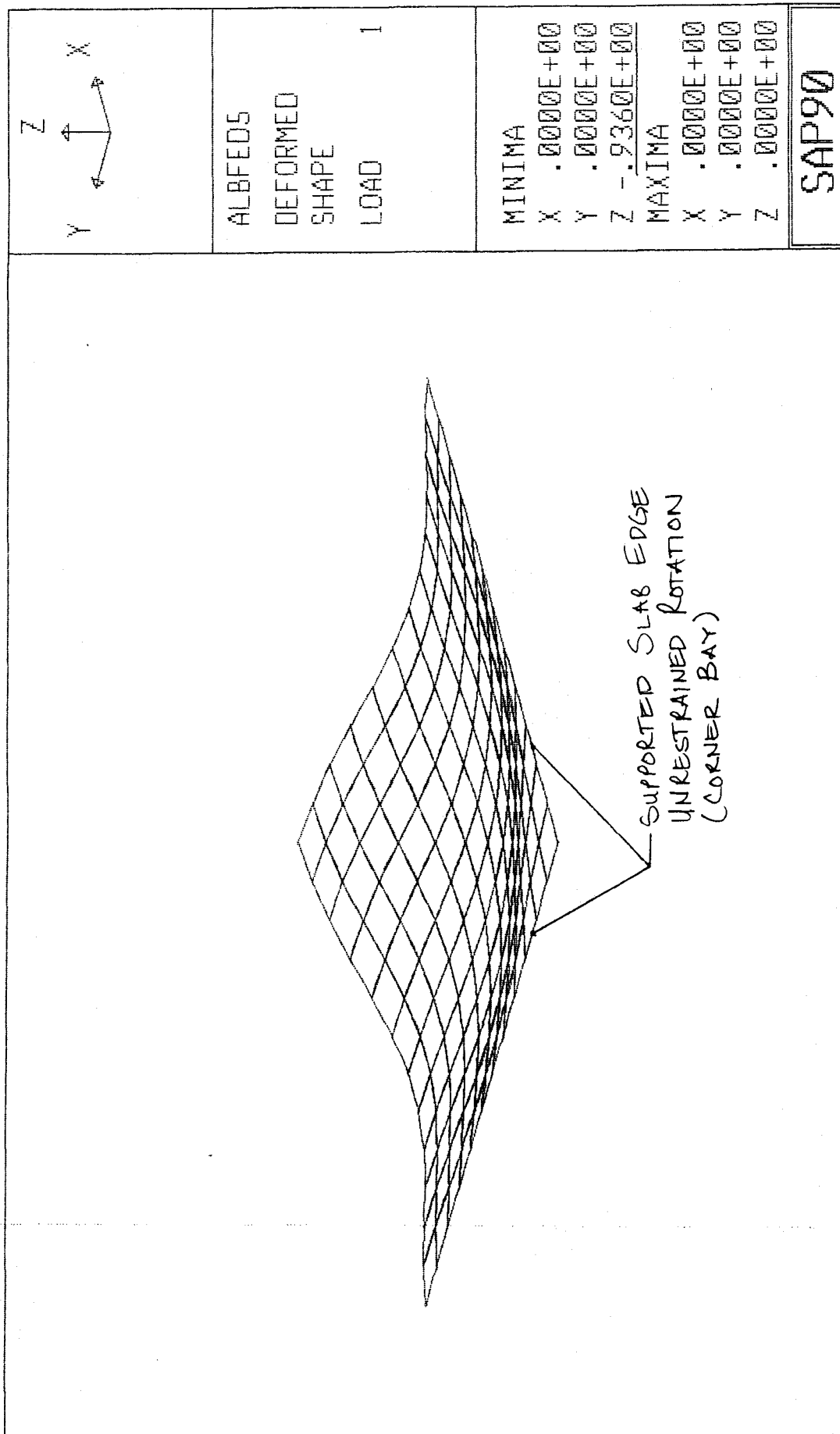


FIGURE 21

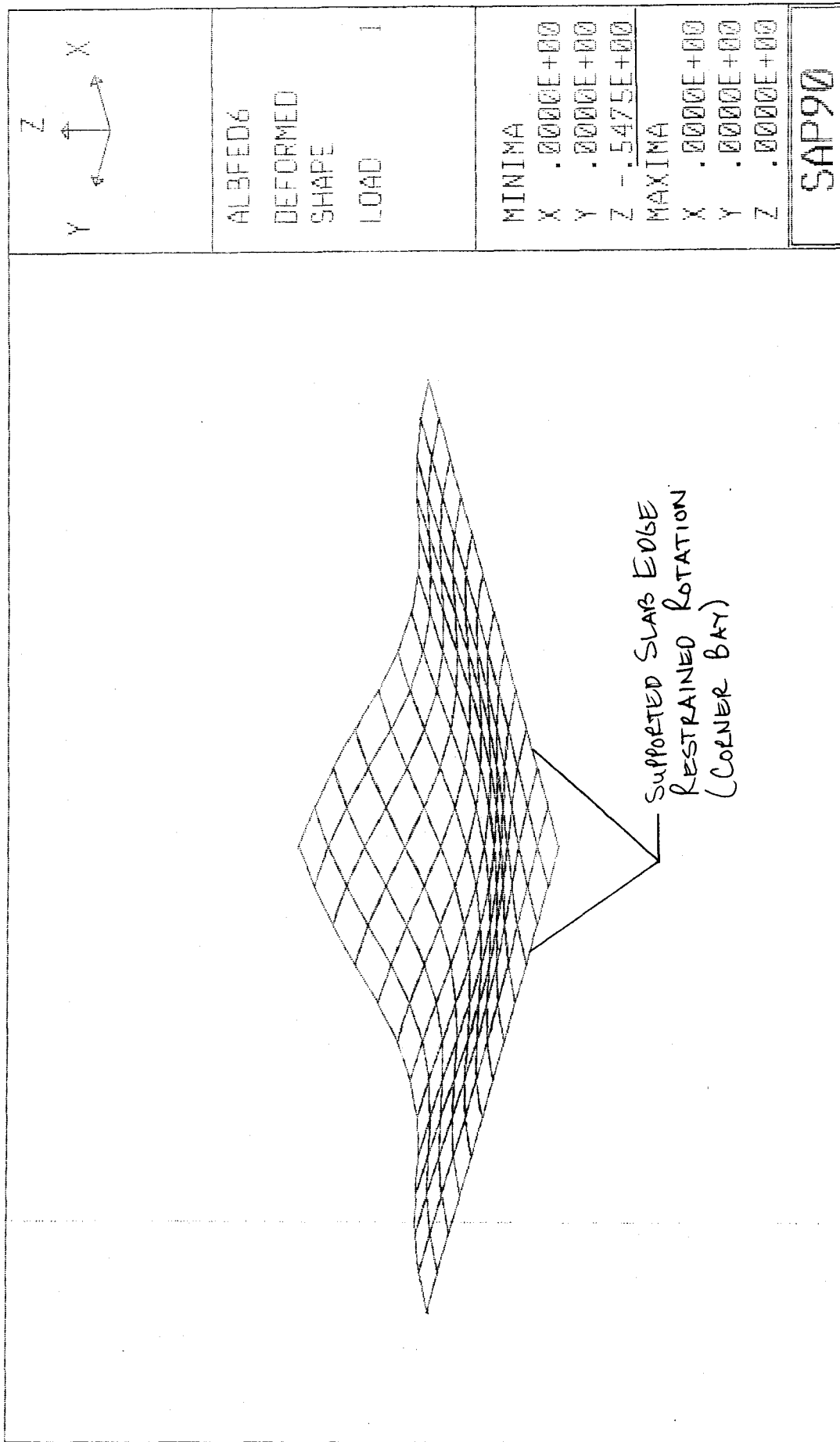


FIGURE 22

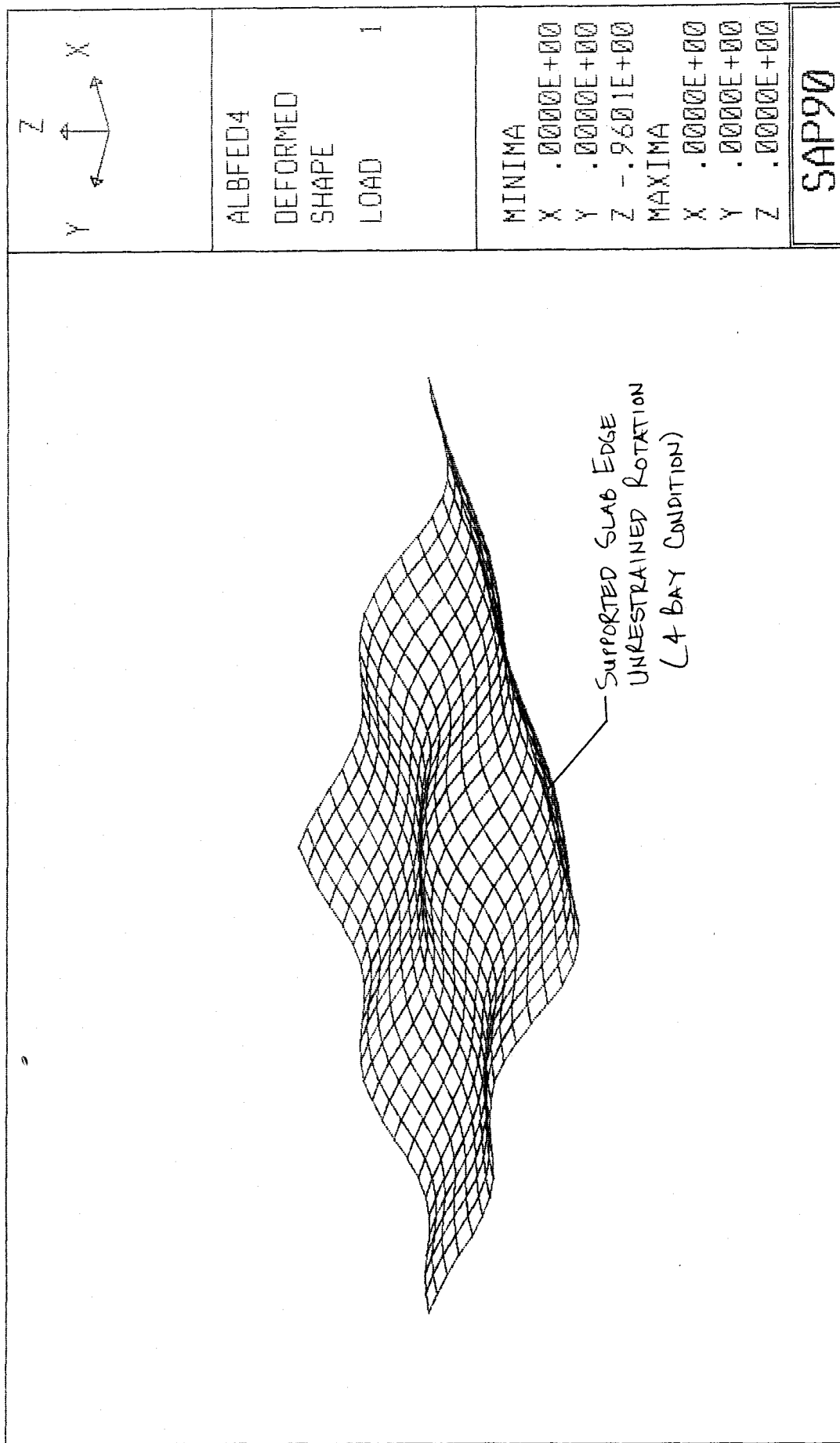


FIGURE 23

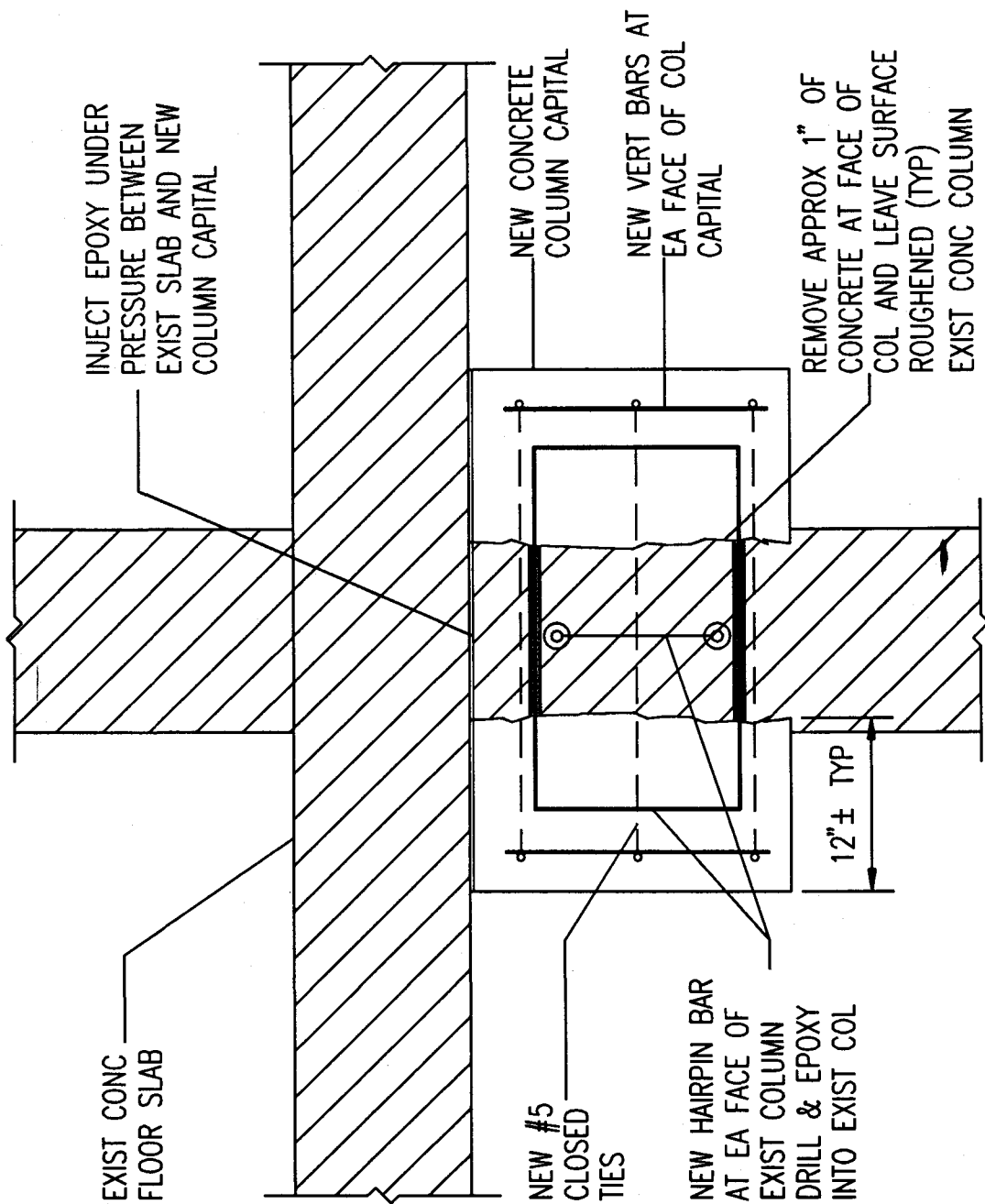
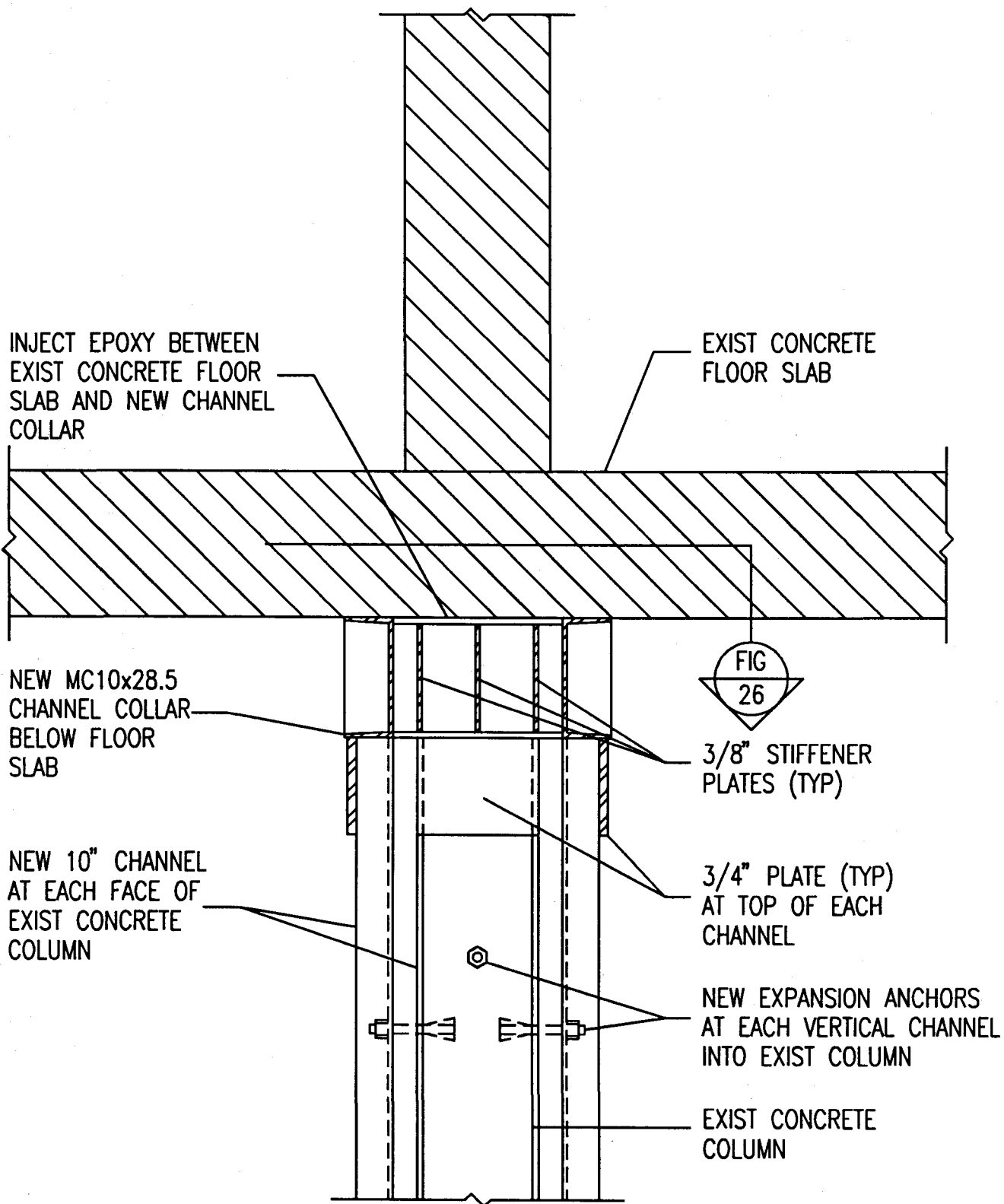


FIGURE 24
ALBUQUERQUE FEDERAL BUILDING



NEW STEEL COLLAR BELOW EXISTING FLOOR SLAB

NO SCALE

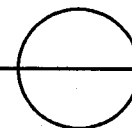
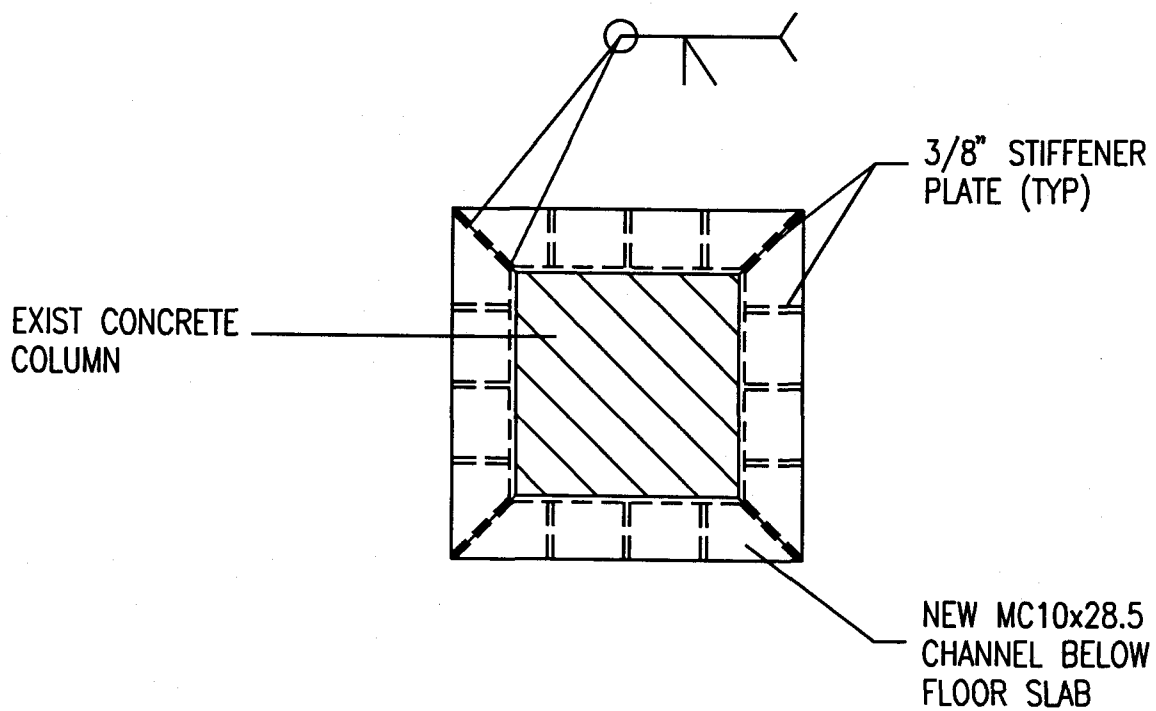


FIGURE 25
ALBUQUERQUE FEDERAL BUILDING



NEW STEEL COLLAR BELOW EXISTING FLOOR SLAB

NO SCALE

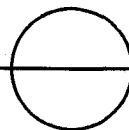
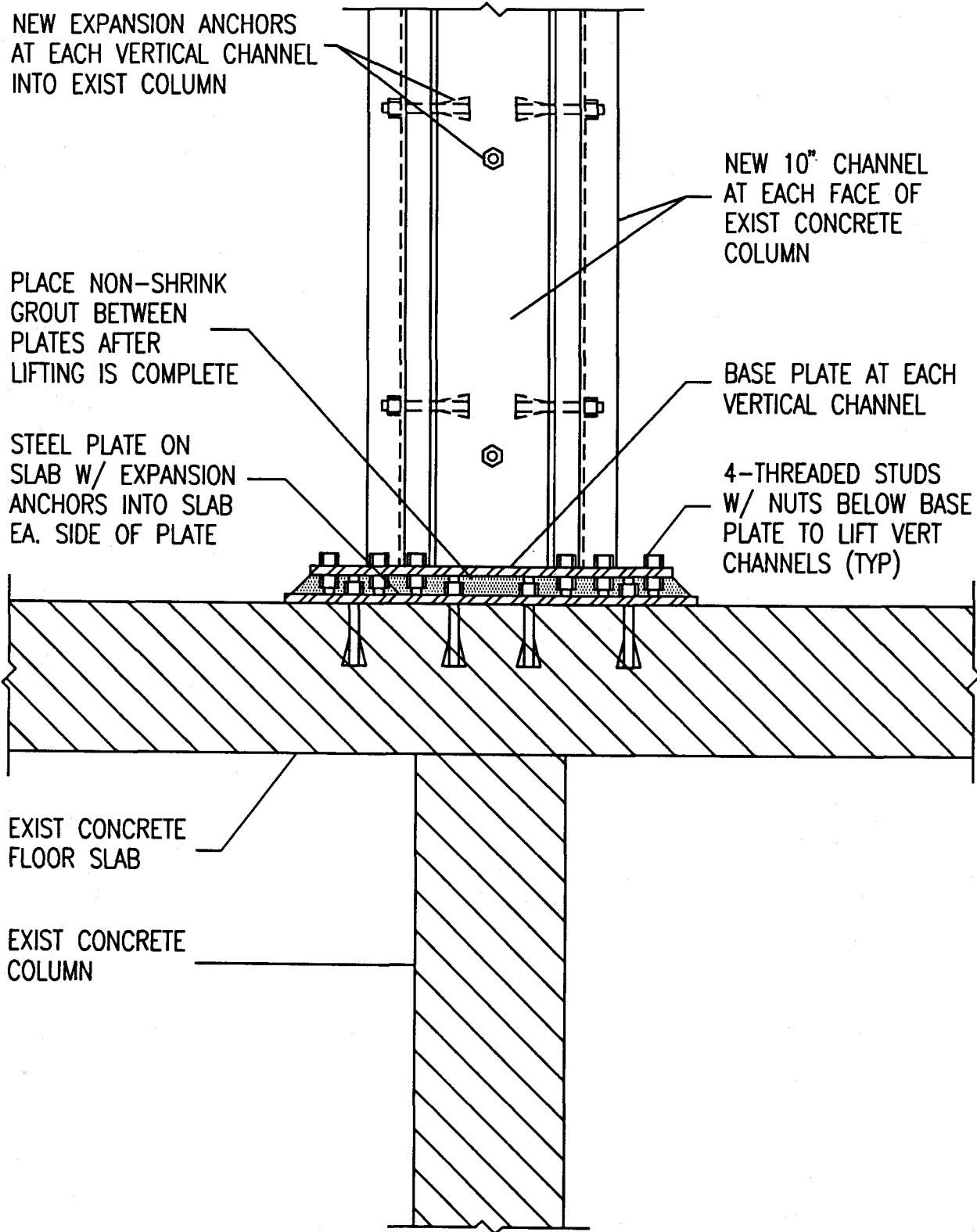


FIGURE 26
ALBUQUERQUE FEDERAL BUILDING



NEW STEEL COLLAR ABOVE EXISTING FLOOR SLAB

NO SCALE

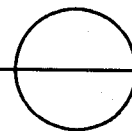
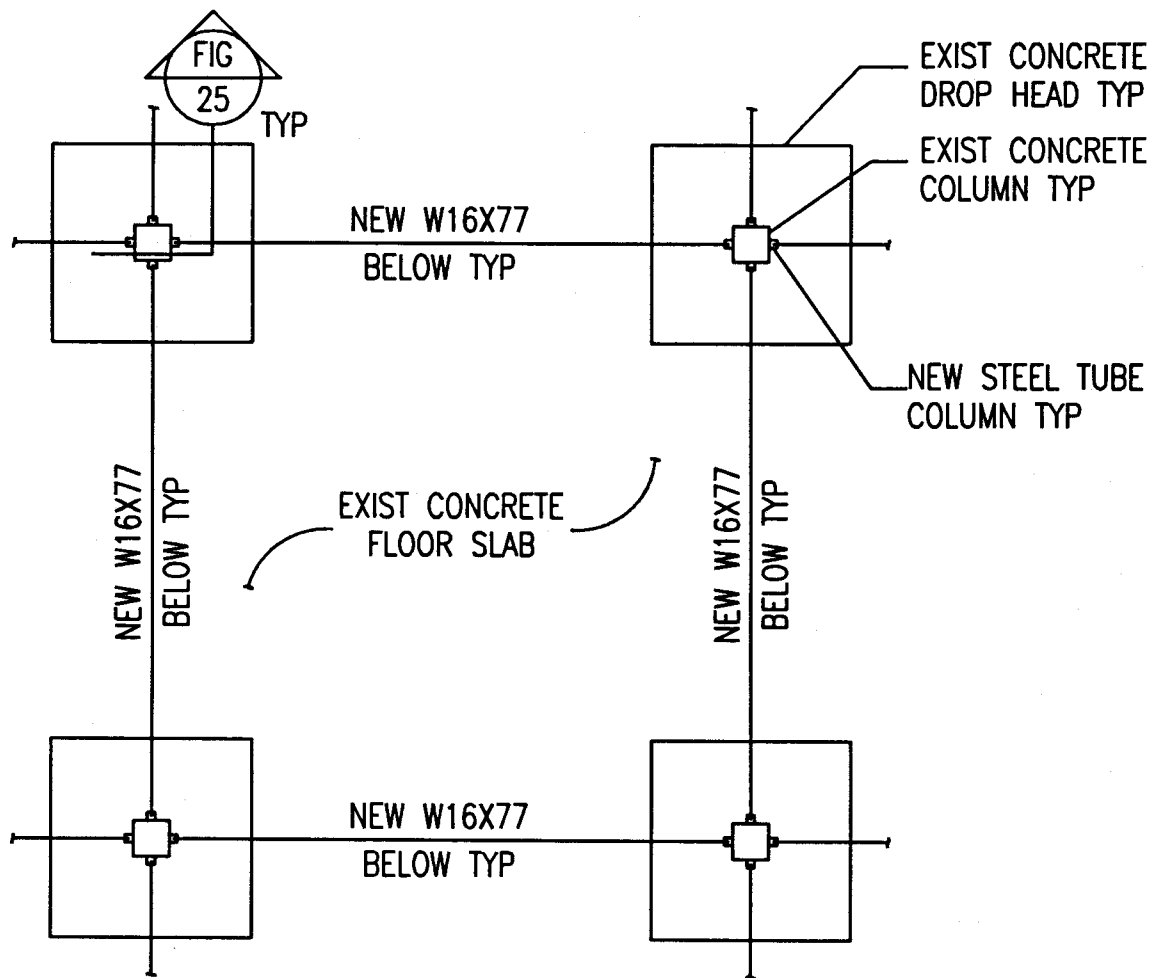


FIGURE 27
ALBUQUERQUE FEDERAL BUILDING



NEW STEEL LIFTING BEAM BELOW EXIST FLOOR SLABS

NO SCALE

FIGURE 28
ALBUQUERQUE FEDERAL BUILDING

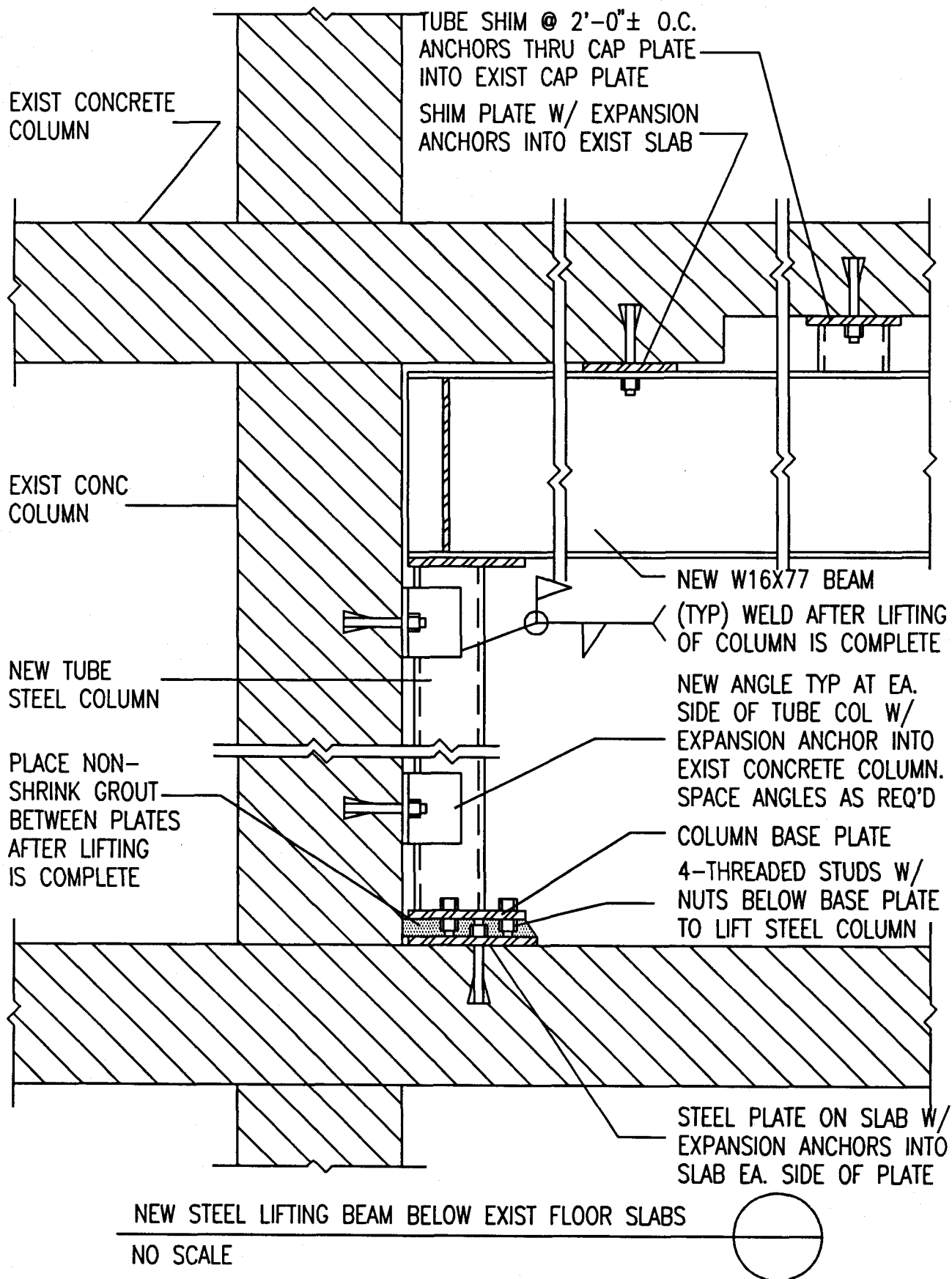
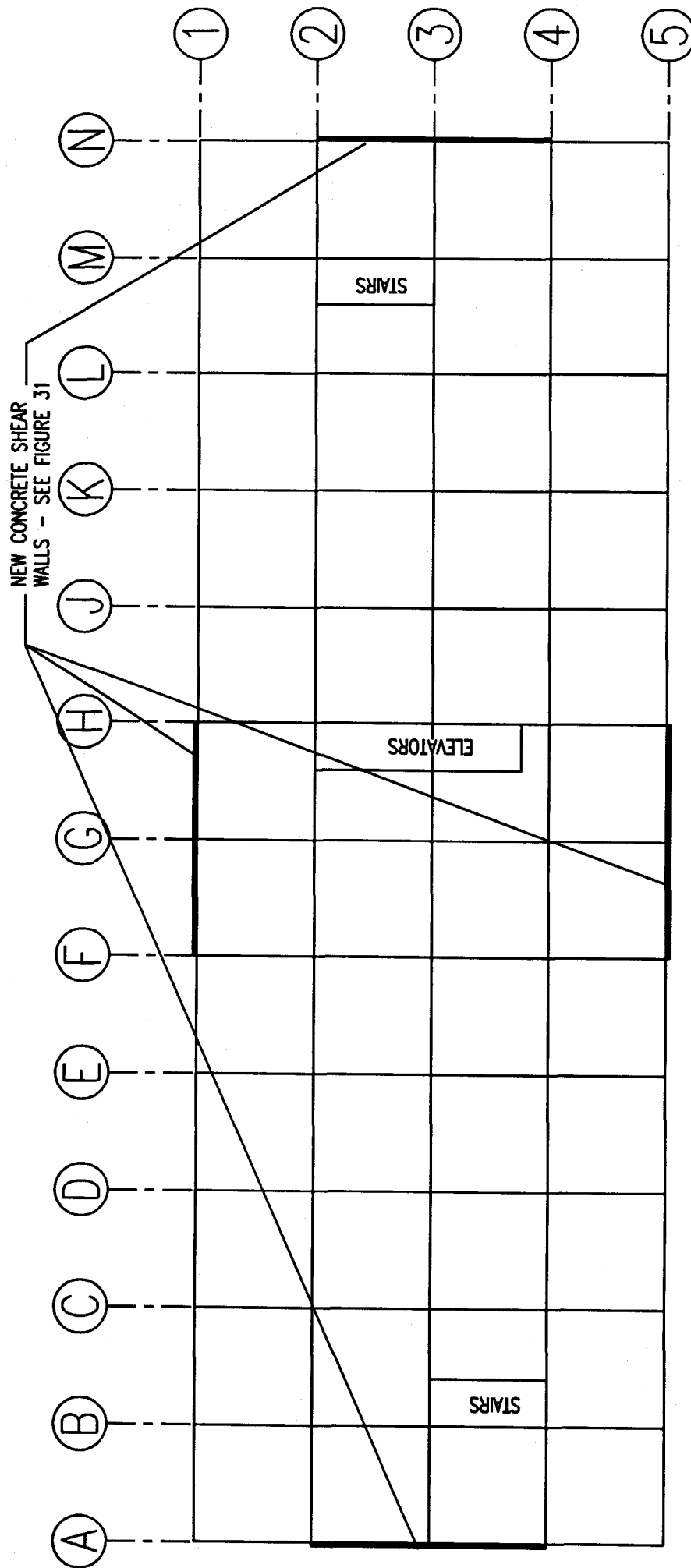
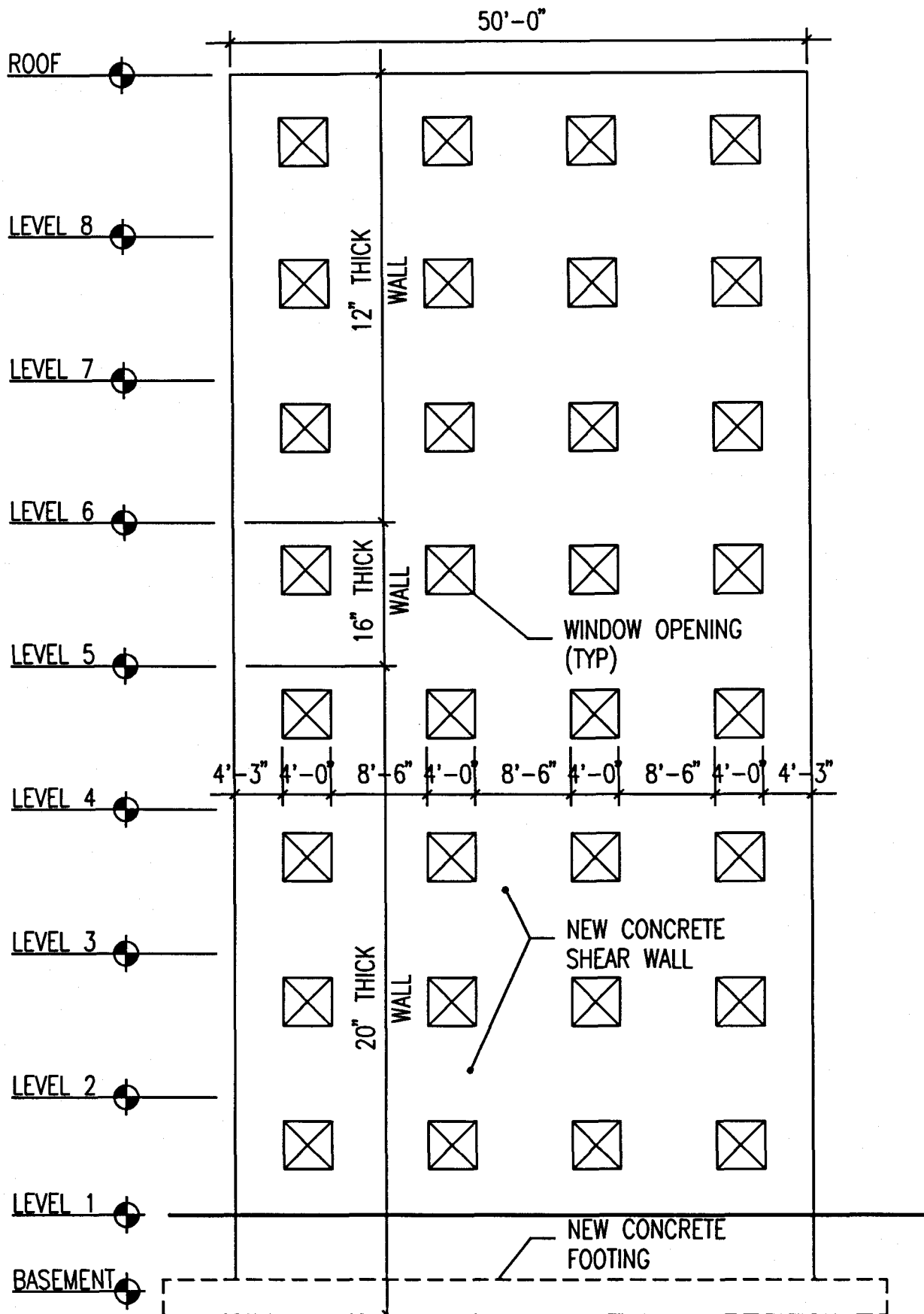


FIGURE 29
ALBUQUERQUE FEDERAL BUILDING

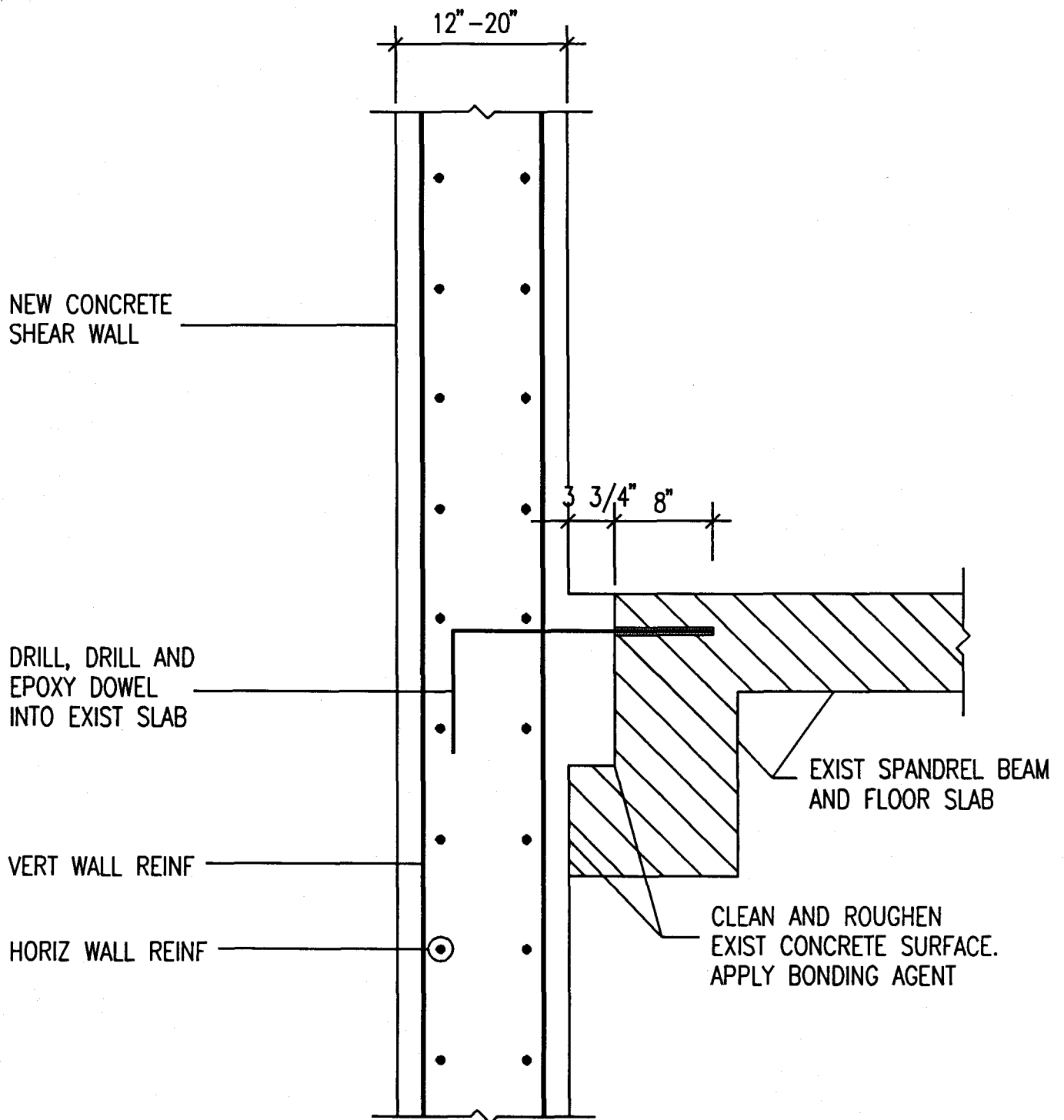


ALBUQUERQUE FEDERAL BUILDING
 LOCATION OF NEW CONCRETE SHEAR WALLS
 FIGURE 30



ALBUQUERQUE FEDERAL BUILDING
NEW CONCRETE SHEAR WALL ELEVATION

FIGURE 31



TYPICAL SHEAR WALL TO FLOOR SLAB CONNECTION

NO SCALE

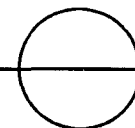
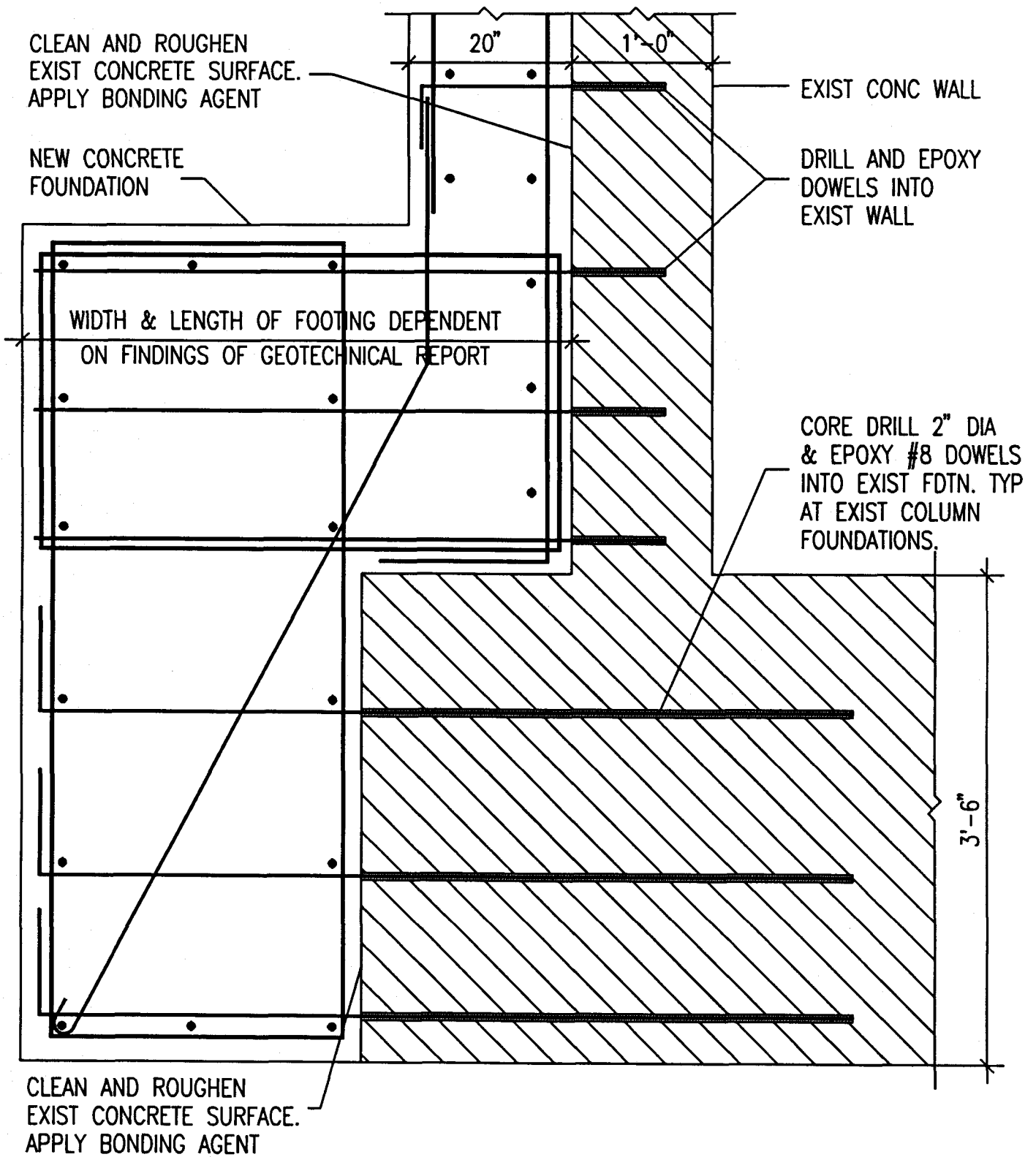


FIGURE 32
ALBUQUERQUE FEDERAL BUILDING



FOUNDATION DETAIL AT EXISTING COLUMN FOOTING

NO SCALE

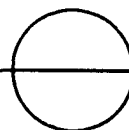


FIGURE 33
ALBUQUERQUE FEDERAL BUILDING

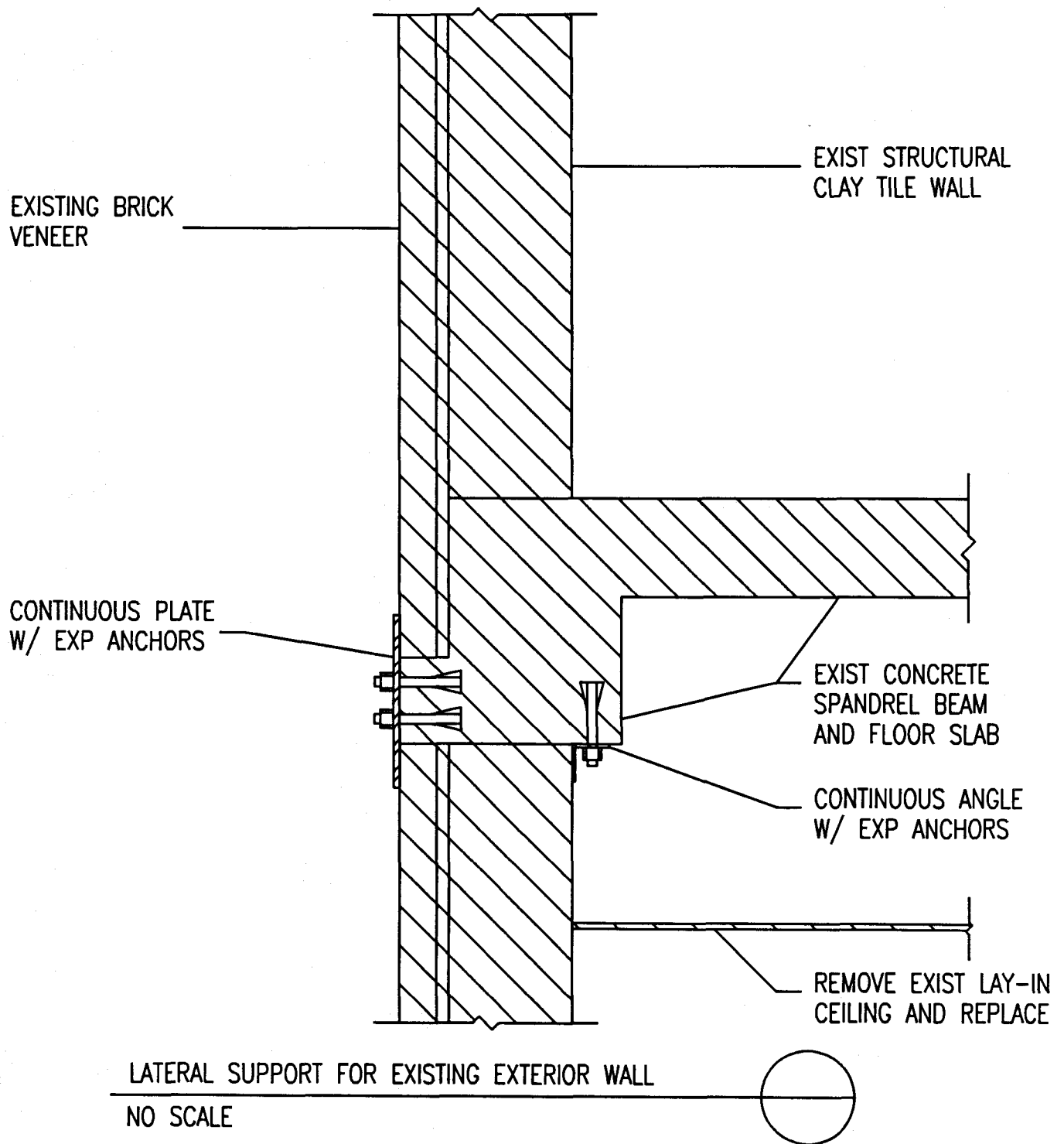


FIGURE 34
ALBUQUERQUE FEDERAL BUILDING

FRAME AT GRIDS D,E & F, AT LEVELS 6,7 & 8
MOMENT COMPARISON

FRAME LOCATION	REQUIRED MOMENT CAPACITY* (FT-K)	ALLOW. MOMENT CAPACITY (FT-K)	ALLOW./REQ'D MOMENT
COLUMN STRIP AT EXTERIOR COLUMN NEGATIVE BENDING	-193.9	-233.1	1.20
COLUMN STRIP AT FIRST INTERIOR COLUMN - NEGATIVE BENDING	-368.5	-381.8	1.04
MIDDLE STRIP AT FIRST INTERIOR COLUMN - NEGATIVE BENDING	-122.8	-140.5	1.14
COLUMN STRIP AT CENTER COLUMN NEGATIVE BENDING	-288.2	-342.2	1.19
MIDDLE STRIP AT CENTER COLUMN NEGATIVE BENDING	-96	-112	1.17
COLUMN STRIP AT EXTERIOR SPAN POSITIVE BENDING	137.8	195.2	1.42
MIDDLE STRIP AT EXTERIOR SPAN POSITIVE BENDING	91.8	159.8	1.74
COLUMN STRIP AT INTERIOR SPANS POSITIVE BENDING	105.8	183.8	1.74
MIDDLE STRIP AT INTERIOR SPANS POSITIVE BENDING	70.6	120.8	1.71

* AT 80 PSF LIVE LOAD WITH 30 PSF SUPERIMPOSED DEAD LOAD
ALL MOMENTS LISTED ARE FACTORED MOMENTS

TABLE 1

**FRAME AT GRID 2, AT LEVELS 6,7 & 8
MOMENT COMPARISON**

FRAME LOCATION	REQUIRED MOMENT CAPACITY* (FT-K)	ALLOW. MOMENT CAPACITY (FT-K)	ALLOW./REQ'D MOMENT
COLUMN STRIP AT EXTERIOR COLUMN NEGATIVE BENDING	-172.4	-224.9	1.30
COLUMN STRIP AT FIRST INTERIOR COLUMN - NEGATIVE BENDING	-346.2	-391.2	1.13
MIDDLE STRIP AT FIRST INTERIOR COLUMN - NEGATIVE BENDING	-115.4	-125.5	1.09
COLUMN STRIP AT INTERIOR COLUMN NEGATIVE BENDING	-310.6	-391.2	1.26
MIDDLE STRIP AT INTERIOR COLUMN NEGATIVE BENDING	-103.5	-125.5	1.21
COLUMN STRIP AT EXTERIOR SPAN POSITIVE BENDING	126.7	180.9	1.43
MIDDLE STRIP AT EXTERIOR SPAN POSITIVE BENDING	84.5	136.2	1.61
COLUMN STRIP AT INTERIOR SPANS POSITIVE BENDING	112.8	180.9	1.60
MIDDLE STRIP AT INTERIOR SPANS POSITIVE BENDING	75.2	136.2	1.81

* AT 80 PSF LIVE LOAD WITH 30 PSF SUPERIMPOSED DEAD LOAD
ALL MOMENTS LISTED ARE FACTORED MOMENTS

TABLE 2

**FRAME AT GRIDS D,E&F, AT LEVELS 2,3,4&5
MOMENT COMPARISON**

FRAME LOCATION	REQUIRED MOMENT CAPACITY* (FT-K)	ALLOW. MOMENT CAPACITY (FT-K)	ALLOW./REQ'D MOMENT
COLUMN STRIP AT EXTERIOR COLUMN NEGATIVE BENDING	-226	-256.1	1.13
COLUMN STRIP AT FIRST INTERIOR COLUMN - NEGATIVE BENDING	-341.2	-362.3	1.06
MIDDLE STRIP AT FIRST INTERIOR COLUMN - NEGATIVE BENDING	-113.7	-126.4	1.11
COLUMN STRIP AT CENTER COLUMN NEGATIVE BENDING	-272.7	-321.6	1.18
MIDDLE STRIP AT CENTER COLUMN NEGATIVE BENDING	-90.9	-112	1.23
COLUMN STRIP AT EXTERIOR SPAN POSITIVE BENDING	125.1	172	1.37
MIDDLE STRIP AT EXTERIOR SPAN POSITIVE BENDING	83.4	134.2	1.61
COLUMN STRIP AT INTERIOR SPANS POSITIVE BENDING	95.7	159.8	1.67
MIDDLE STRIP AT INTERIOR SPANS POSITIVE BENDING	63.8	120.8	1.89

* AT 80 PSF LIVE LOAD WITH 30 PSF SUPERIMPOSED DEAD LOAD
ALL MOMENTS LISTED ARE FACTORED MOMENTS

TABLE 3

**FRAME AT GRID 2, AT LEVELS 2,3,4&5
MOMENT COMPARISON**

FRAME LOCATION	REQUIRED MOMENT CAPACITY* (FT-K)	ALLOW. MOMENT CAPACITY (FT-K)	ALLOW./REQ'D MOMENT
COLUMN STRIP AT EXTERIOR COLUMN NEGATIVE BENDING	-201.6	-250.4	1.24
COLUMN STRIP AT FIRST INTERIOR COLUMN - NEGATIVE BENDING	-327.3	-346.6	1.06
MIDDLE STRIP AT FIRST INTERIOR COLUMN - NEGATIVE BENDING	-109.1	-108.7	1.00
COLUMN STRIP AT INTERIOR COLUMN NEGATIVE BENDING	-296.1	-346.6	1.17
MIDDLE STRIP AT INTERIOR COLUMN NEGATIVE BENDING	-98.7	-108.7	1.10
COLUMN STRIP AT EXTERIOR SPAN POSITIVE BENDING	115.9	166.4	1.44
MIDDLE STRIP AT EXTERIOR SPAN POSITIVE BENDING	77.3	136.2	1.76
COLUMN STRIP AT INTERIOR SPANS POSITIVE BENDING	100.6	166.4	1.65
MIDDLE STRIP AT INTERIOR SPANS POSITIVE BENDING	67	136.2	2.03

* AT 80 PSF LIVE LOAD WITH 30 PSF SUPERIMPOSED DEAD LOAD
ALL MOMENTS LISTED ARE FACTORED MOMENTS

TABLE 4

SLAB DEFLECTIONS USING ADOSS OUTPUT

LOCATION OF SLAB DEFLECTION	SHORT-TERM DEFLECTION	ADDITIONAL LONG-TERM DEFLECTION (2 X SHORT-TERM)	TOTAL DEFLECTION
INTERIOR BAY - BETWEEN COLUMNS	0.43**	0.86"	1.29" = .11'
INTERIOR BAY - MIDDLE OF BAY	0.64"	1.28"	1.92" = .16'
EXTERIOR BAY - BETWEEN COLUMNS	0.53**	1.06"	1.59" = .13'
EXTERIOR BAY - MIDDLE OF BAY	0.665"	1.33"	1.995" = .17'
CORNER BAY - BETWEEN COLUMNS	0.52**	1.04"	1.56" = .13'
CORNER BAY - MIDDLE OF BAY	0.64"	1.28"	1.92" = .16'

* AVERAGE VALUE

TABLE 5

SLAB DEFLECTIONS USING ADOSS OUTPUT

LOCATION OF SLAB DEFLECTION	SHORT-TERM DEFLECTION	CREEP DEFLECTION	SHRINKAGE DEFLECTION	TOTAL DEFLECTION
INTERIOR BAY - BETWEEN COLUMNS	0.43**	0.62"	0.29"	1.34" = .11'
INTERIOR BAY - MIDDLE OF BAY	0.64"	0.92"	0.29"	1.85" = .16'
EXTERIOR BAY - BETWEEN COLUMNS	0.53**	0.76"	0.29"	1.58" = .13'
EXTERIOR BAY - MIDDLE OF BAY	0.665"	0.96"	0.29"	1.92" = .16'
CORNER BAY - BETWEEN COLUMNS	0.52**	0.75"	0.29"	1.56" = .13'
CORNER BAY - MIDDLE OF BAY	0.64"	0.92"	0.29"	1.85" = .16'

* AVERAGE VALUE

TABLE 6

SLAB DEFLECTIONS USING FINITE ELEMENT OUTPUT

LOCATION OF SLAB DEFLECTION	SHORT-TERM DEFLECTION	ADDITIONAL LONG-TERM DEFLECTION (2 X SHORT-TERM)	TOTAL DEFLECTION
INTERIOR BAY - BETWEEN COLUMNS	0.454"	0.908"	1.362" = .11'
INTERIOR BAY - MIDDLE OF BAY	0.72"	1.44"	2.16" = .18'
EXTERIOR BAY - BETWEEN COLUMNS	0.574**	1.147"	1.72" = .14'
EXTERIOR BAY - MIDDLE OF BAY	0.782**	1.564"	2.346" = .20'
CORNER BAY - BETWEEN COLUMNS	0.576**	1.152"	1.728" = .14'
CORNER BAY - MIDDLE OF BAY	0.742**	1.484"	2.226" = .19'

* AVERAGE VALUE

TABLE 7

SLAB DEFLECTIONS USING FINITE ELEMENT OUTPUT

LOCATION OF SLAB DEFLECTION	SHORT-TERM DEFLECTION	CREEP DEFLECTION	SHRINKAGE DEFLECTION	TOTAL DEFLECTION
INTERIOR BAY - BETWEEN COLUMNS	0.454"	0.654"	0.29"	1.40" = .12'
INTERIOR BAY - MIDDLE OF BAY	0.72"	1.04"	0.29"	2.05" = .17'
EXTERIOR BAY - BETWEEN COLUMNS	0.574**	0.827"	0.29"	1.69" = .14'
EXTERIOR BAY - MIDDLE OF BAY	0.782**	1.126"	0.29"	2.20" = .18'
CORNER BAY - BETWEEN COLUMNS	0.576**	0.829"	0.29"	1.695" = .14'
CORNER BAY - MIDDLE OF BAY	0.742**	1.068"	0.29"	2.10" = .18'

* AVERAGE VALUE

TABLE 8

WEST WALL

LEVEL	ALLOWABLE FORCE (KIPS)	CALCULATED FORCE (KIPS)	PERCENT STRESSED
ROOF	368	124	34%
8 TH	368	224	61%
7 TH	368	309	84%
6 TH	368	380	103%
5 TH	368	439	119%
4 TH	368	485	132%
3 RD	368	527	143%
2 ND	368	524	142%

EAST WALL

LEVEL	ALLOWABLE FORCE (KIPS)	CALCULATED FORCE (KIPS)	PERCENT STRESSED
ROOF	368	40	11%
8 TH	368	99	27%
7 TH	368	168	46%
6 TH	368	226	61%
5 TH	368	279	76%
4 TH	368	331	90%
3 RD	368	402	109%
2 ND	368	468	127%

PERCENT STRESSED IS BASED UPON SHEAR ONLY. WALLS HAVE LITTLE
IF ANY BOUNDARY REINFORCING FOR RESISTING OVERTURNING.

**ALBUQUERQUE FEDERAL BUILDING
WEST STAIRWAY WALL SHEAR STRESSES**

TABLE 9

WEST WALL

LEVEL	ALLOWABLE FORCE (KIPS)	CALCULATED FORCE (KIPS)	PERCENT STRESSED
ROOF	627	531	85%
8 TH	627	682	109%
7 TH	627	822	131%
6 TH	627	936	149%
5 TH	627	1011	161%
4 TH	627	1032	165%
3 RD	627	990	158%
2 ND	627	791	126%

PERCENT STRESSED IS BASED UPON SHEAR ONLY. WALLS HAVE LITTLE IF ANY BOUNDARY REINFORCING FOR RESISTING OVERTURNING.

**ALBUQUERQUE FEDERAL BUILDING
ELEVATOR SHAFT WALL SHEAR STRESSES**

TABLE 10

WEST WALL

LEVEL	ALLOWABLE FORCE (KIPS)	CALCULATED FORCE (KIPS)	PERCENT STRESSED
ROOF	368	354	96%
8 TH	368	381	104%
7 TH	368	426	116%
6 TH	368	459	125%
5 TH	368	469	127%
4 TH	368	441	120%
3 RD	368	435	118%
2 ND	368	431	117%

EAST WALL

LEVEL	ALLOWABLE FORCE (KIPS)	CALCULATED FORCE (KIPS)	PERCENT STRESSED
ROOF	368	338	92%
8 TH	368	383	104%
7 TH	368	433	118%
6 TH	368	473	129%
5 TH	368	490	133%
4 TH	368	466	127%
3 RD	368	372	101%
2 ND	368	405	110%

PERCENT STRESSED IS BASED UPON SHEAR ONLY. WALLS HAVE LITTLE
IF ANY BOUNDARY REINFORCING FOR RESISTING OVERTURNING.

**ALBUQUERQUE FEDERAL BUILDING
EAST STAIRWAY WALL SHEAR STRESSES**

TABLE 11